This is a digital copy of a book that was preserved for generations on library shelves before it was carefully scanned by Google as part of a project to make the world's books discoverable online.

It has survived long enough for the copyright to expire and the book to enter the public domain. A public domain book is one that was never subject to copyright or whose legal copyright term has expired. Whether a book is in the public domain may vary country to country. Public domain books are our gateways to the past, representing a wealth of history, culture and knowledge that's often difficult to discover.

Marks, notations and other marginalia present in the original volume will appear in this file - a reminder of this book's long journey from the publisher to a library and finally to you.

Usage guidelines

Google is proud to partner with libraries to digitize public domain materials and make them widely accessible. Public domain books belong to the public and we are merely their custodians. Nevertheless, this work is expensive, so in order to keep providing this resource, we have taken steps to prevent abuse by commercial parties, including placing technical restrictions on automated querying.

We also ask that you:

+ *Make non-commercial use of the files* We designed Google Book Search for use by individuals, and we request that you use these files for personal, non-commercial purposes.
+ *Refrain from automated querying* Do not send automated queries of any sort to Google's system: If you are conducting research on machine translation, optical character recognition or other areas where access to a large amount of text is helpful, please contact us. We encourage the use of public domain materials for these purposes and may be able to help.
+ *Maintain attribution* The Google "watermark" you see on each file is essential for informing people about this project and helping them find additional materials through Google Book Search. Please do not remove it.
+ *Keep it legal* Whatever your use, remember that you are responsible for ensuring that what you are doing is legal. Do not assume that just because we believe a book is in the public domain for users in the United States, that the work is also in the public domain for users in other countries. Whether a book is still in copyright varies from country to country, and we can't offer guidance on whether any specific use of any specific book is allowed. Please do not assume that a book's appearance in Google Book Search means it can be used in any manner anywhere in the world. Copyright infringement liability can be quite severe.

About Google Book Search

Google's mission is to organize the world's information and to make it universally accessible and useful. Google Book Search helps readers discover the world's books while helping authors and publishers reach new audiences. You can search through the full text of this book on the web at `http://books.google.com/`

BRANNER
GEOLOGICAL LIBRARY

STANFORD UNIVERSITY LIBRARIES

DEPARTMENT OF THE IN

WATER-SUPPLY

AND

IRRIGATION P

OF THE

UNITED STATES GEOLOGICAL

No. 18

WASHINGTON
GOVERNMENT PRINTING OFFI
1898

BRANNER
GEOLOGICAL LIBRARY

STANFORD UNIVERSITY LIBRARIES

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 18

UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

IRRIGATION NEAR FRESNO, CALIFORNIA

BY

CARL EWALD GRUNSKY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1898

659116

CONTENTS.

	Page.
Letter of transmittal	9
Kaweah River	11
Hydrography	11
Canals and ditches	14
Wutchumna Canal	14
Curtis & Lindsey Ditch	16
Venice Ditch (North Side Rocky Ford Ditch)	16
Elbow Ditch No. 1	16
Matthew Ditch, or Elbow Ditch No. 2	16
Curtis Ditch	17
Weston Ditch	17
Uphill Ditch	17
Settlers Ditch	17
Lakeside Ditch	18
Jennings Ditch	20
Rodgers Ditch	20
Modoc Canal	20
Hicks & Weston Ditch	20
Hayes Upper Ditch (Goshen Ditch)	20
Hayes Lower Ditch	21
Pogues Upper Ditch	21
Pogues Lower Ditch	21
Ogden Ditch	21
Hamilton Ditch	22
Consolidated Peoples Ditch	22
Swanson Ditch	25
Lee & Thompson Ditch	25
Rice Ditch	25
Catron and Teague ditches	25
Bliss & Hyde Ditch	25
Davis, Brown, and Pennebaker ditches	25
Extension Ditch	25
Kaweah Canal	26
Farmers Canal	27
Tulare Canal	28
Rocky Ford Ditch	29
Cameron Creek Company	29
Bacon & Crossmore Canal (Packwood Creek Canal)	29
Judd Ditch	30
Cutler & Higgins Ditch (Chatten Ditch)	31
Kaweah and Mill Creek Water Company	31
Oaks Ditch	31
Evans Ditch	32
Evans & Turner Ditch	32
Burch Ditch	32

Kaweah River—Continued. Page.
Canals and ditches—Continued.
Watson Ditch 32
Mill Ditch 32
Bahwell and Fulgham ditches 33
Longs Canal 33
Bostwick Cut 33
Ketchum Cut 34
Crowley & Mehrtens Ditch 34
Tulare irrigation district 34
Water supply, soils, and methods of irrigation 35
Kings River 39
Hydrography 39
Canals and ditches 42
Kings River & Fresno Canal 42
Fresno Canal 43
Centerville Ditch 47
Sweem Ditch 48
Fowler Switch Canal 48
Centerville & Kingsburg Canal 50
Selma irrigation district 52
Alta irrigation district (76 Canal) 52
Morrow Ditch 57
Dennis & Jordan Ditch 57
Kincaid Ditch 57
Glenn and Barton ditches 57
Emigrant Canal 57
Liberty Canal 58
Millrace Canal 58
Reeds Ditch 58
Laguna de Tache Canal (Grant Canal) 58
Crescent Canal 62
Peoples Ditch 62
Mussel Slough Ditch 64
Last Chance Ditch 65
Lower Kings River Canal 67
Rhoads Canal 69
Jacobs Canal 69
Sunset irrigation district 69
The Fresno plains 71
Description 71
The canal system 73
Loss of water by evaporation and seepage 74
Effect of irrigation on the water table 78
Methods of irrigating and duty of water 79
Wildflower 85
Liberty and Millrace settlements 86
Laguna de Tache Rancho 86
Mussel Slough country 87
Description 87
Canal system 87
Method of irrigation 88

ILLUSTRATIONS.

Page.

PLATE I.—Map showing canals from Kaweah and Tule rivers 11
II.—*A*, Flume over Tulare Canal; *B*, Main canal of Tulare irrigation district 28
III.—*A*, Canal regulator, Tulare irrigation district; *B*, Lateral, 1 mile south of Tulare 34
IV.—Map of east side of San Joaquin Valley, from Kings River to Fresno River 38
V.—Kings River at upper end of Centerville Bottoms, looking upstream 42
VI.—*A*, Regulator on Kings River & Fresno Canal; *B*, Fowler Switch Canal near Sanger, looking upstream 44
VII.—Cobblestone and brush dam of Fresno Canal, Kings River 46
VIII.—Regulator of Fresno Canal on Kings River: *A*, As seen from above; *B*, As seen from below 48
IX.—Regulator on Fowler Switch Canal: *A*, As seen from above; *B*, As seen from below 50
X.—Regulator on 76 Canal: *A*, As seen from above; *B*, As seen from below 52
XI.—*A*, Furrow irrigation for citrus fruits; *B*, Check weir in Alta irrigation district, in lateral, about 2 miles west of Traver 56
XII.—Lower Kings River canal system 58
XIII.—Peoples Ditch: *A*, Brush dam at time of high water; *B*, Regulator, as seen from above 62
XIV.—Last Chance Canal regulator: *A*, As seen from upper side; *B*, As seen from below 66
FIG. 1.—Head of Wutchumna Canal 15
2.—Head of Uphill Ditch 17
3.—Head of Lakeside Ditch 19
4.—Head of Consolidated Peoples Ditch 22
5.—Ditch system at head of Kaweah Canal 26
6.—Head of Farmers Canal 28
7.—Head of Bacon & Crossmore Canal 30
8.—Head of Bacon & Crossmore Cut 30
9.—Discharge of Kings River at Red Mountain, California, 1896 41
10.—Head of Kings River & Fresno Canal 42
11.—Head of Fresno Canal 44
12.—Head of Fowler Switch Canal 49
13.—Head of Centerville & Kingsburg Canal 51
14.—Wahtoke Reservoir, on 76 Canal 53
15.—Head of 76 Canal 54
16.—Plan and elevation of regulator for 76 Canal 55
17.—Head of Grant Canal 60
18.—Head of Peoples Canal, on Kings River 63
19.—Head of Last Chance Ditch, on Kings River 66
20.—Upper section of Lower Kings River Canal 68
21.—Irrigation by filling subsoils with water, Mussel Slough country 89

LETTER OF TRANSMITTAL.

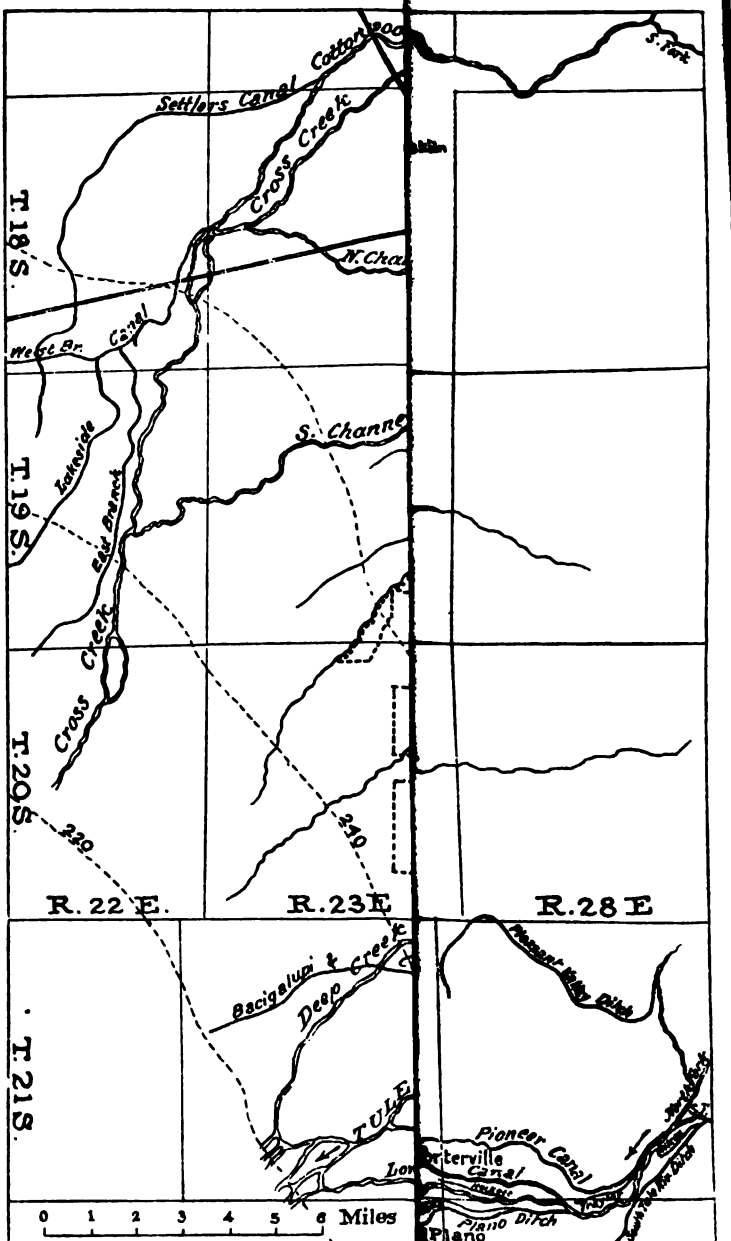
DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, July 6, 1898.

SIR: I have the honor to transmit herewith a manuscript entitled Irrigation near Fresno, California, by Mr. C. E. Grunsky, and to recommend that it be published as one of the series of pamphlets on Water Supply and Irrigation. This is the second of three papers relating to irrigation in San Joaquin Valley, the other two being Nos. 17 and 19 of the series. It relates mainly to the irrigation systems deriving their water supply from Kaweah and Kings rivers, and covering the agricultural lands on the east side of San Joaquin Valley, between Visalia and Fresno.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.


Settlers Canal
Cotton
Cross Creek
S. Fork
T. 18 S.
N. Cha
West Br.
Canal
Lakeside
East Branch
S. Channe
T. 19 S.
Cross Creek
T. 20 S.
220
240
R. 22 E.
R. 23 E
R. 28 E
Bacigalupi &
Deep Creek
Pleasant Valley Ditch
T. 21 S.
TULE
Pioneer Canal
Canal
Plano Ditch
Plano
0 1 2 3 4 5 6 Miles

IRRIGATION NEAR FRESNO, CALIFORNIA.[1]

By C. E. GRUNSKY.

KAWEAH RIVER.

HYDROGRAPHY.

Kaweah River enters San Joaquin Valley from the east, passing between Dillons Point on the north end and Wutchumna Hill on the south, about 15 miles east of Visalia. Its drainage basin above that point has an area of 619 square miles, mostly low mountain country. The river has a perennial flow, but its flow is comparatively small at low stages, ordinarily about 30 second-feet. The low-water period usually begins in August and continues through January. The river is subject to freshets in the winter months, and is at its highest stages in April, May, and June. Medium stages are expected throughout February, March, and July. The monthly mean flow at the river's highest stage exceeds 4,000 second-feet.[2]

Before the freshets of 1861–62, which have become memorable, the river annually overflowed large areas in its course across the east-side valley plain, and numerous high-water channels carried its waters toward Tulare Lake. The portion of its overflowed district to the east of or above Visalia is generally referred to as Visalia Swamp. Four of the river channels—Elbow Creek, Visalia or Mill Creek, Packwood Creek, and Deep Creek—gave the name of "Four Creek Country" to the vicinity of Visalia. The channel at the northern edge of the overflowed land was known as Canoe Creek above Iron Mountain, and as Elbow Creek below that point. The course of Elbow Creek is north of west to a junction with Cottonwood Creek, a small water course from the northeast, thence southwesterly to a reunion with Kaweah River water in Cross Creek, which name is given to the lower portion

[1] For an account of irrigation in other parts of San Joaquin Valley, see Water-Supply and Irrigation Papers Nos. 17 and 19. These three papers are based largely upon information collected by the State engineering department of California ten years ago, and are intended to supplement the two volumes issued by that department, namely: Irrigation Development, History, Customs, Laws, etc., in France, Italy, and Spain, by Wm. Ham. Hall, C. E., State Engineer, Sacramento, California, 1886; and Irrigation in California (Southern), Part 2 of Report of Wm. Ham. Hall, C. E., State Engineer of California 1888. (See introductory paragraph on page 13 of Water-Supply and Irrigation Paper No. 17.)

[2] Physical Data and Statistics, Wm. Ham. Hall, State Engineer of California, pp. 455, 476.

of St. Johns Channel of the river. Deep Creek was the principal south marginal stream. It received water from the main channel of the river 8 miles east of Visalia and lost its identity about 5 miles southeast of Visalia, where its waters spread and were received into other channels of different names. Visalia Creek and Packwood Creek were the names given to the two water courses into which the main channel of Kaweah River separated 5 miles above Visalia. The course of Visalia Creek was westerly through Visalia, that of Packwood Creek southwesterly. To understand the changes made in the system of principal channels of the river by the high waters of 1862 and the effects of these changes, it is necessary to know that, even preceding that year, irrigation and drainage work had been done in the Kaweah River delta. The Shipp Cut had been made in 1854. It was a small drain ditch from the swamp, near where Rocky Ford now is, southwesterly to a connection with Canoe Creek. A canal had been constructed from Visalia Creek into Visalia to supply power to a flour mill. The Lander Ditch, the Watson Ditch, and a number of other small ditches from Visalia and Packwood creeks had also become dependent upon the flow of water in the river channels at the south of Visalia. The freshets of 1861–62 cut a new channel from a point on Kaweah River about 14 miles east of Visalia in a northwesterly direction toward and along the northern border of Kaweah River Swamp. Shipp Cut and a section of Canoe Creek were enlarged by the flood waters and became a part of this new channel, and finally a connection was established with the Cross Creek channel below Visalia, and thus St. Johns Channel of Kaweah River was permanently established. At the same time a new high-water channel was washed out toward the northeast from a point on Kaweah River 6 miles above Visalia to a connection with this new channel. This is known as Sand Slough, which still remains as a connection between the two river channels at a point where they are only about one-half mile apart. The same high waters caused so much erosion in Landers Ditch that it became a branch of the river, and is generally referred to as Landers Slough. Its upper portion was long in use as the channel through which the Consolidated Peoples Ditch received its water.

The upper sections of the original main channel of Kaweah River were obstructed by the drift and silt deposits of these freshets, and as the low-water period of 1862 approached, it was found that water was scarce for those depending upon Deep Creek, Packwood Creek, and Visalia Creek for their supply, and many projects were proposed for relief. During the years immediately following, a number of ditches were constructed from the St. Johns Channel of the river, near Rocky Ford, southwesterly to the original main channel. Such were Longs Canal, Ketchum Canal, and the Bostwick Cut. All of these were intended to increase the flow in the delta streams at and southward of Visalia. After much contention between settlers on these several streams as to

the apportionment of their scant supply of water, it was agreed, in 1867, that a gate should be constructed in the head of Visalia Creek. This gate was not, however, maintained long. The freshets of 1867–68 were almost equal to those of 1861–62 and effected further enlargement of the St. Johns Channel. They refilled and otherwise destroyed some of the ditches that had been opened as connections between the two rivers. They washed out the gate in the head of Visalia Creek, and partially closed the heads of Packwood and Deep creeks. They enlarged Sand Slough. At the same time the upper portions of the old channel of Kaweah River were further obstructed by drift and silt, and a new head of St. Johns Channel was eroded about a mile above the former point of separation of the two channels.

To overcome the increased difficulties of keeping a good supply of water in those channels of the river reaching the vicinity of Visalia, many ineffectual efforts were made and much money was almost uselessly expended until, in 1877, a contract was entered into with Mr. Samuel Fowler by the Kaweah Canal and Irrigation Company, the Consolidated Peoples Ditch Company being an interested party, to open the old channel of the river. The cut then made, practically following the old channel, was enlarged and deepened by erosion, until a few years thereafter it was again the low-water channel of Kaweah River. Another obstructed section of the old channel near the head of the Consolidated Peoples Ditch, about 10 miles east of Visalia, was improved about the same time by the Kaweah and Mill Creek Water Company, and has since been known as the Visalia Cut or Hamilton Cut.

The obstructed head of Packwood Creek was improved by the Rocky Ford Canal Company, but was not in service long. This creek now receives its principal supply of water through the Bacon & Crossmore Cut from Visalia Creek. It has a southwesterly course, and such of its water as is not used for irrigation is delivered into Tulare Lake.

Cross Creek has already been mentioned as receiving the water of St. Johns Channel, Elbow Creek, and Cottonwood Creek. After approaching within 4 miles of Traver its course becomes southwesterly to a union with some of the old high-water delta channels of Kings River. Its course is thence southerly along the eastern border of this delta to Tulare Lake, which receives its water.

Visalia or Mill Creek separates into two channels soon after passing through Visalia. The course of one of these is a little north of west, that of the other southwest. Both unite with Cross Creek before reaching Tulare Lake.

Cameron Creek is one of the lesser delta channels of Kaweah River. It receives a portion of the flow of Deep Creek at a point about 6 miles due east from Visalia, and has a southwesterly course to Tulare Lake. It passes just to the northward of Tulare. An artificial channel at the head of this water way, constructed to improve its connection with Deep Creek, was closed by an order of court some years ago.

Deep Creek has already been mentioned. Its waters at the spread or sink of the creek were conducted by small artificial channels into Lower Deep Creek and Bates Slough. These artificial channels were soon so enlarged by erosion that the two lower sloughs have become extensions of the Deep Creek channel and conduct its waters onward southwesterly toward Tulare Lake.

Outside Creek has a course along the eastern margin of the Kaweah delta. The creek can not be traced to a direct connection with the present river channels. It may, in fact, be regarded as the extension into the river delta of Yokohl Creek, a small stream draining a portion of the foothill section intermediate between the Kaweah River and Tule River drainage basins; but in times of flood the natural flow of some of the river water was into depressions, reaching a point that may be regarded as the head of Outside Creek, about 9 miles east of Visalia. The creek now receives an occasional inflow of foothill drainage waters from the east and the waste water from some of the branches of the Consolidated Peoples Ditch. The water of Outside Creek is distributed to a number of channels, the most southerly of which is Elk Bayou, which unites southeast of Tulare with the old channel of Tule River, and thence has a southwesterly course to a union with the present main channel of Tule River.

CANALS AND DITCHES.

Wutchumna Canal.—This is the upper north-side diversion of water from Kaweah River. The head of the canal is at the base of the slope of Dillons Point, a foothill spur rising to the northward. Thence it has a westerly course for 4 miles to a flat depression, which its water converts into a lake of about 40 acres, known as Bravo Lake. Leaving Lake Bravo, the westerly course of the canal is maintained 4 miles farther to the northern base of Iron Mountain, and thence 6 miles southwesterly to St. Johns Channel, about 4 miles above Visalia, where its water is carried over the river channel in a flume. The main canal then flows westerly near the south bank of the river about 2 miles to a junction with Old or Lower Wutchumna Ditch, whose branches extend far to the west of Visalia.

The Wutchumna Water Company was incorporated in 1872 for the avowed purpose of effecting a better distribution of water to the several ditches dependent upon Kaweah River for their supply, but various difficulties were encountered which prevented the enterprise from meeting with anticipated success, and it was not until 1880 that a connection between the upper diversion—work on which is reported to have commenced in 1872—and the distributing system of the lower canal was finally established. The lower canal was constructed in 1873 or 1874. Its head is on the south side of the St. Johns Channel of Kaweah River, about 3 miles northeast from Visalia. It established a direct southerly connection between St. Johns River and Visalia Creek,

and for three years its water furnished power for the flour mill in Visalia. The principal branch of the Lower Wutchumna Canal has a westerly course, passing just to the north of Visalia and terminating at a point about 4 miles south from Goshen.

The first work at the head of the Wutchumna Canal seems to have been done by Stephen Barton, who subsequently appears as one of the directors in the Pioneer Canal Company, which was incorporated in 1877, and which during that season constructed a ditch on the line adopted by the Wutchumna Canal as far as Bravo Lake. When the Wutchumna Canal Company, as successor of the Pioneer Company, assumed control, this upper ditch was extended along the route already described.

The canal head gate or regulator has been placed in a rock cut, and, as a result of this location of the head of the canal, expenditures of maintenance became unusually heavy when, a few years after the canal came into use, it became necessary to cut the upper section of the canal 4 feet deeper. This was done at a cost of $6,000. The total cost of canal construction has probably exceeded $60,000. The head gate width is 20 feet, and this may be regarded as the average bed width of the canal throughout its principal section. Its fall is reported to be nearly 7 feet to the mile for 4 miles, thence 2½ feet to the mile. The flume across St. Johns River is 14 feet wide, 2 feet deep, and about 150 feet long. It was built in 1880. The head gate of the Lower Wutchumna Canal is a little over 11 feet wide.

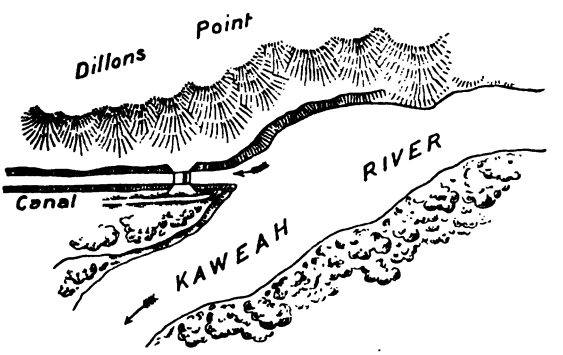


FIG. 1.—Head of Wutchumna Canal.

For a distance of about 4 miles westward from Lake Bravo the canal follows the original alignment of the Curtis & Lindsey Ditch. This ditch was enlarged, and its owners were granted a right to use that section of the Wutchumna Canal.

The canal is owned by an incorporated company whose shares are all in the hands of about thirty persons. All canal expenses are met by assessment of the stockholders. Each stockholder is entitled to a proportional part of the water in the canal, and he may dispose of this at his pleasure. He may lease this to whom he pleases, but not in less amounts than full shares. The distribution of water is in charge of a canal superintendent, who apportions it to the various distributing ditches in proportion to the number of shares represented in each. Although the company owns some of the shares of stock, these are not

for sale, nor will the company lease them or the water which they represent. By retaining these the company increases the proportional part of water represented by each of the other shares. The canal company bears the expense of the necessary structures at the heads of distributing ditches, but does not keep them in repair or replace them if destroyed. This has to be done under direction of the canal superintendent by the parties directly interested in their maintenance.

Curtis & Lindsey Ditch.—This ditch has its head on the north side of St. Johns River, 4 miles below the head of the Wutchumna Canal. It has a northwesterly course for one-half mile to a connection with the Wutchumna Canal, into which it drops its water. Four miles below this point the water dropped into the Wutchumna Canal is again taken out by a ditch 4 miles long having a northwesterly course and terminating on the bottom lands of Cottonwood Creek. The Curtis & Lindsey Ditch was constructed in 1876, and its extension toward Cottonwood Creek was made in the following year. Soon afterwards 4 miles of its course were adopted as the alignment for the Wutchumna Canal, so that the ditch now appears in two sections, of which the lower one may also be considered a branch of the Wutchumna Canal. The ditch is also known as the Curtis, Lindsey & Moffitt Ditch. Its width was about 10 feet on the bottom, and its fall between 2 and 3 feet to the mile, but, so far as known, the extent of irrigation from it has not exceeded several hundred acres.

Venice Ditch.—This, called also North Side Rocky Ford Ditch, is a small north-side ditch which seems to have been first used about 1873. It was extended and enlarged somewhat in 1877. Its head is at Rocky Ford. It has a westerly course to Iron Mountain, thence skirts the eastern and southern base of this mountain, and terminates 2 miles farther west. It is about 6 miles long and 3 to 8 feet wide. Ownership in the ditch is represented by shares held by the farmers who use its water.

Elbow Ditch No. 1.—The flow of Elbow Creek, which leaves St. Johns River at a point about 5 miles above Visalia, is controlled, in a measure at least, by a regulator or head gate 8 feet wide, and is frequently referred to as Elbow Ditch No. 1. The upper part of the creek is an enlargement by erosion of a portion of the ditch known as Shipps Cut, which was constructed in 1854.

Matthew Ditch, or Elbow Ditch No. 2.—This is a small ditch having its head on the north side of St. Johns River, about 2 miles below the head of Elbow Creek. It was constructed in 1855, but with a somewhat different alignment from the present. The freshets of 1861–62 so changed the position and character of Kaweah River that the head of the ditch had to be carried upstream. It is difficult to determine whether the ditch remained in continuous use. The head gate was put into the new ditch in 1875 and was set somewhat lower than the bed of the river. A few years later erosion had progressed in

the river channel to such extent that a wing dam had to be used to divert water into the ditch. The ditch has a northwesterly course, is about 6 miles long, 6 feet wide on the bottom, carries water about 1½ feet deep, and has a grade of about 6 feet to the mile. It irrigates between 200 and 300 acres, of which nearly one-half is alfalfa.

Curtis Ditch.—About 1879 there was a small ditch in use occasionally for the irrigation of a few acres of alfalfa on the north side of St. Johns River about a mile below the head of the Matthew Ditch. It was known as the Curtis Ditch, and has fallen into disuse.

Weston Ditch.—Another abandoned small ditch which was in use for a few seasons prior to 1880 on the north side of the river, about a mile above the head of the Uphill Ditch, was the Weston Ditch.

Uphill Ditch.—This is another north-side ditch. Its head is half a mile above the road leading northward from Visalia. Diversion is made from the river without the use of any permanent weir. The diverted water is led in a cut a few hundred yards northwesterly and is dropped into a natural depression or high-water channel, in which it continues on the same course half a mile. Thence it is taken in the ditch proper northerly about 3 miles to the place of use in the vicinity of Elbow Creek. The head gate is 8 feet wide. It is in the form of a box culvert, and bears a heavy fill of sand on top, intended to give it stability. The sand deposits in the cut to the river above the head gate have made it somewhat expensive to keep the head of the ditch open. The ditch was constructed in 1871. It was made 8 feet wide on the bottom, and was intended to carry water 2 feet deep. Its fall is 20 inches to the mile.

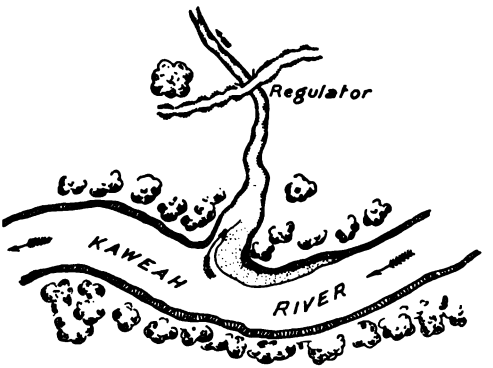


FIG. 2.—Head of Uphill Ditch.

Settlers Ditch.—This is a canal which for many years was of considerable importance as a source of water supply for lands in the eastern part of the Kings River delta. The canal receives water through a branch of Cross Creek. The canal regulator is on the north side of that creek 3 miles south of Traver. The creek channel, there tapped by the canal, is really the extension of Cottonwood and Elbow creeks, but most of its flow comes through a cut, the real head of Settlers Ditch, a quarter of a mile long, leading from the main channel of Cross Creek to its northern branch, at a point about 4 miles above the Settlers Canal head gate. Where the canal is finally diverted from Cross Creek a light timber weir or waste gate has been placed across the

creek channel. The canal regulator is nearly half a mile below the point where the canal leaves the natural channel. The head gate is 32 feet wide. It has eight openings, each 3 feet 6 inches wide in the clear. The gates closing these openings are provided with stems to which power is applied by means of levers.

The canal proper has a bed width at and near its head of 32 feet, and can carry water 2½ feet deep. It has 9 inches fall to the mile, and a capacity of 75 to 80 second-feet. The width of the canal is reduced within 3 miles of its head gate to 24 feet on the bottom, and its fall to 6 inches to the mile. Its course is a little south of west for 5 miles to a point where its water is divided, being carried thence southerly in two channels, of which the easterly branch is about 4½ miles long, and the westerly about 7 miles. The ditch commands a territory of about 8,000 acres, of which as much as 5,000 acres have been benefited by its water in a single season. The canal construction commenced in 1874, and considerable land was irrigated in 1876. Water was available only during the high stages of Kaweah River.

There was but little irrigation with water from this canal at any time to the east of the Kings River delta lands, soils being too alkaline to encourage cultivation. The extent to which this canal is now in use is somewhat problematical, as all the rights to water which it was supposed to have acquired were sold to the Tulare irrigation district, which has made a diversion from the St. Johns River about one-half mile below Rocky Ford. The canal is still in service, and does not appear to have suffered great diminution in its supply of water during the high-water stage of the river. The Settlers Ditch Company, which built the Settlers Canal at a first cost of approximately $35,000, was incorporated in 1874 with a capital stock of $10,000, in 50 shares of $200 each. These were held, at the time of the sale of the canal water rights, by about 50 persons.

All expenses of construction, repairs, maintenance, and management have been met by assessing the stockholders. Each share of stock entitles its holder to a proportional part of the water in the canal. Water is apportioned to each branch and to each distributing canal in proportion to the number of shares for which each draws water. No water is sold by the canal company, nor does the company lease any of the several shares which have reverted to it. Each stockholder, however, is permitted to lease his water to other persons, and can have it delivered to any point of the main canal or its branches which he selects. Water is delivered to irrigators through gates of various forms, generally under pressure. Its equitable distribution depends to a great extent on the judgment of the superintendent. Each share of water is supposed to be sufficient to irrigate 160 acres. The cost of water to the irrigators has averaged about 70 cents per acre per annum.

Lakeside Ditch.—This is another important diversion from the lower

portion of Cross Creek. The head of this canal is 4 miles below the head gate of the Settlers Canal, and 6 miles west from Goshen. The diversion from the natural creek channel is accomplished, as in the case of the Settlers Canal, by means of a weir or waste gate placed in the creek channel below the head of the canal, and a regulator or head gate in the latter about one-half mile below its head. The weir is a light timber structure arranged as a series of gates. It has a total width of 41 feet. Its nine openings between vertical posts are each 4 feet wide in the clear. The canal head gate has a width of 31 feet. The space between side walls is divided by vertical posts into six openings, each about 4½ feet wide in the clear. These openings or spaces between posts are closed with vertically sliding gates, to the stems of which power is applied by means of levers. The canal has a southwesterly course from its head for about 3 miles. In this distance it has a bed width of 30 feet and a fall of 6 inches to the mile. It can carry water about 3 feet deep. Its branches extend in southerly and southwesterly directions to the vicinity of Tulare Lake. The main canal and principal branches have an aggregate length of about 50 miles and command about 20,000 acres of land. The cultures on at least as much as one-half of this area may be considered to have been benefited by irrigation from this canal in a single season. The canal was constructed in 1874 and the years immediately following, at a total first cost of about $35,000.

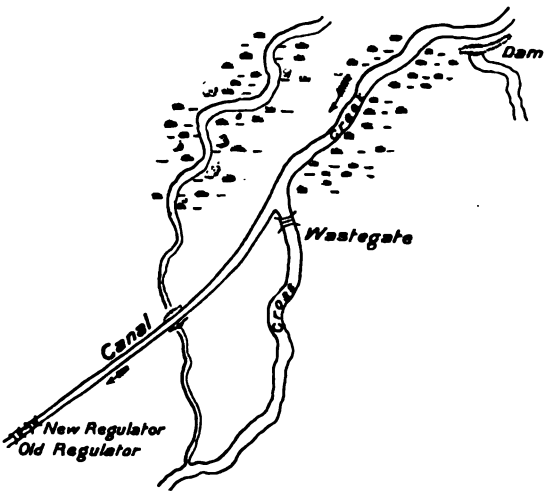


FIG. 3.—Head of Lakeside Ditch.

The canal is owned by the Lakeside Ditch Company, which was incorporated on August 7, 1874, with a capital stock of $10,000, divided into 10,000 shares of $1 each. On February 20, 1875, the capital stock was increased to 30,000 shares of $1 each. These shares were owned by ninety-five persons in 1885. Each share of stock entitles its holder to a proportional share of water in the canal. The canal company does not sell or rent water, not even on the few shares of stock held in the company's name. Each stockholder can have his water delivered at any point of any branch of the company's canal system. He can, if he desires, rent or sell his water for one or more

seasons. The company originally intended to deliver to each stockholder the water to which he was entitled at some point on his own land, but this system of delivery was found to be inexpedient and was abandoned in 1876. The distribution of water is in charge of a canal superintendent, who apportions the water to the various branches of the canal and has control of its delivery to the irrigator. The average annual cost of water from this canal has been about $1.55 per acre.

Jennings Ditch.—This is a small ditch which has its head on the south side of St. Johns River a little more than a quarter of a mile below the head of the Lower Wutchumna Ditch, and sends its branches into the northern portion of Visalia and adjacent territory. The main ditch has a bed width of 10 feet. The diversion of water is accomplished by means of a light wing dam of sand and brush, and the flow of water in the ditch is controlled by means of a simple regulator in the form of a box, 10 feet wide, which can be closed with loose flashboards. Water rights of an old date, preceding the formation of St. Johns River, are claimed for this ditch, whose head has been shifted back and forth between Mill Creek and St. Johns River.

Rodgers Ditch.—This is a small ditch, about 4 feet wide on the bottom, which supplies water to a few acres of land north and west of Visalia. It was constructed about 1875.

Modoc Canal.—This canal or ditch was originally intended to be made a distributary for the Wutchumna Canal water. It was constructed in 1876. Its head is on the south side of St. Johns River, just above the road leading northward from Visalia. Its length is about 4 miles, the course being a little south of west. Its bottom width is about 10 feet, and its capacity about 36 second-feet. Ownership in the ditch is represented by shares of stock. Water is distributed to stockholders in proportion to their interests. The annual expense of maintenance is about $100, which covers the construction of an inexpensive wing dam at the head of the ditch. The area actually irrigated is about 1,000 acres.

Hicks & Weston Ditch.—This is a very small ditch from the south side of St. Johns River, about 200 yards above the Modoc Canal. The ditch is 4 to 5 feet wide on the bottom. It was constructed in 1879. It receives water from the Wutchumna Canal in addition to its independent supply, and is sometimes called the Hicks Branch of Wutchumna Canal.

Hayes Upper Ditch (Goshen Ditch).—This ditch was constructed in 1881 or 1882 to supplement an older small ditch which was constructed in 1874 for the irrigation of lands near Goshen, theretofore considered almost worthless on account of the abundance of alkaline salts in the soil. The upper ditch has its head on the south side of St. Johns River about 4 miles northwest of Visalia. It has a westerly course and is about 4 miles long. Its dimensions are irregular. The pro

moters of this enterprise constructed a small ditch and gave it an inexpensive, poorly protected head gate. The gate was carried off by the first freshet and the ditch was rapidly enlarged by erosion, so that it became a canal of considerable capacity. The second regulator put into its head was also a very light structure, 18 feet wide. Like many of the structures on Kaweah River, it was arranged as a box culvert, the entrance to which was divided into nine openings by vertical posts. The spaces between posts could be closed with loose flashboards. This ditch and the Lower Hayes Ditch both belong to the same owner, who has supplied some water to his neighbors at agreed rates per acre.

Hayes Lower Ditch.—The head of this ditch is about 1½ miles below the head of the Hayes Upper Ditch. Its course is a little south of west to the same region watered by the Hayes Upper Ditch. The ditch is about 4 miles long and 6 feet wide. Its head gate has a width of 10 feet. This ditch was constructed in 1874 by settlers near Goshen, but soon fell into the hands of the promoters of the Goshen Canal enterprise.

Pogues Upper Ditch.—This is the uppermost south-side diversion from Kaweah River. The head of the ditch is about 4 miles above the head of St. Johns River, a mile above the hill known as Limekiln Point. The ditch, which has a bed width of about 6 feet, skirts the northern and western base of Limekiln Point and commands several thousand acres of fine land lying between projecting spurs of the foothills. The ditch was constructed in 1884. Water is carried in a flume for some distance from the head of the ditch. This ditch now serves the citrus orchards at Lemon Cove, which have been set out within the last four or five years and now have an aggregate area of about 500 acres.

Pogues Lower Ditch.—This ditch was constructed in 1875. It diverts water from the south side of Kaweah River just below Limekiln Point. It has a southwesterly course, a bed width of 6 to 8 feet, and passes to the north and west of Wutchumna Hill, following the eastern border of the main valley and terminating at present at a point about 3 miles southeast of Exeter. Its total length is about 15 miles, and the capacity claimed for it is 30 second-feet. There was but little irrigation from this ditch at the time the second Pogue Ditch was constructed, but demand for water is rapidly increasing. The water is used on the lands set out to citrus fruits at and near Bonnie Brae. The area of citrus fruits now set out, all being dependent on this ditch for water, is 550 acres. Purchasers of land from the owners of this ditch are granted the right to call for water at the rate of 1 miner's inch (about one-fiftieth of a second-foot) to 5 acres. The charge of the water is 8 cents per inch per twenty-four hours. The lands (including the water rights) are sold at $75 per acre.

Ogden Ditch.—This ditch has its head on the south side of Kaweah River, about three-quarters of a mile below the head of Pogues Lower

Ditch. It has a southwesterly course, closely following the river, and terminates near Cottage. It is a very small ditch, about 4½ miles long, and supplies water to a few acres of alfalfa. It is said to be one of the old ditches, the date of its construction being given as 1862. The head of the ditch was not, however, always maintained at the same place, this having been impossible on account of the changes in the alignment and character of the river.

Hamilton Ditch.—This is another small private ditch on the south side of Kaweah River. Its head is about a mile above Cottage. It has a southwesterly course, is about 4 miles long, and about 4 feet wide. It has been in use since 1854. A south-side branch of the Hamilton Ditch is sometimes known as the Dillon Ditch.

Consolidated Peoples Ditch.—The difficulties experienced by irrigators

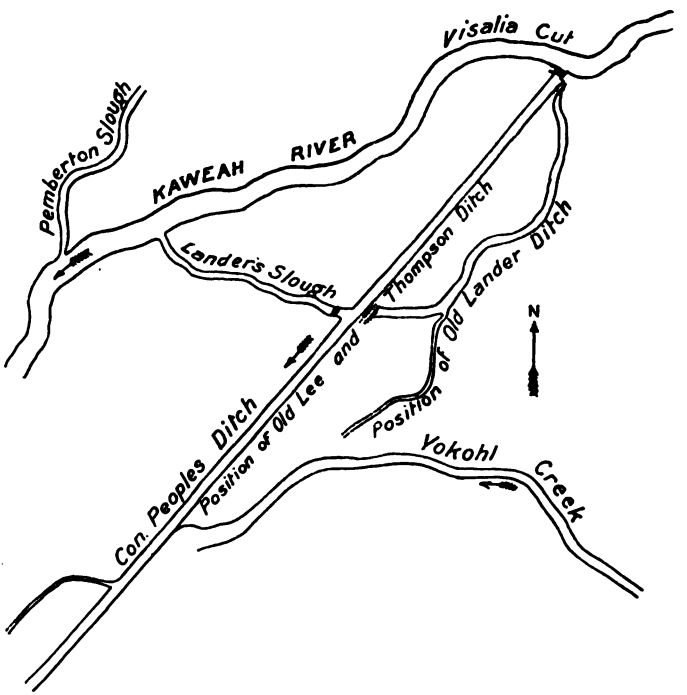


FIG. 4.—Head of Consolidated Peoples Ditch.

dependent for water upon the flow of Deep Creek, Packwood Creek, and Visalia Creek, after the freshets of 1861–62, led to a combination of interests, the direct result of which was the construction of the Peoples Ditch. The outlook for a sufficient supply of water was particularly bad after the winter of 1863–64, in which there was an unusual deficiency of precipitation, and, as a result of a conference with the owners of a number of small irrigation ditches, about one hundred interested persons assembled late in February, 1864, at the point where Landers Slough leaves the river and commenced work on the Peoples

Ditch, the water of which was to be apportioned to a number of ditches which before made independent diversions. The principal ditches that then became dependent upon the Peoples Ditch for water were the Landers, Lee & Thompson, Swanson, Turner, Grubbs, and Rice ditches. The expense of the canal construction was apportioned to the participating farmers according to relative interests. Most of them contributed labor, teams, and supplies. The ditch at its head was made 18 feet wide and 4 feet deep and was to have a fall of about 3½ feet to the mile. Work progressed rapidly and the ditch was considered finished by June 1, 1864. Its estimated cost at that time was $11,000. But owing to the obstructed condition of the old channel of Kaweah River below the head of St. Johns Channel, expectations as to flow of water were not fully realized, and upon petition the water commissioners of Tulare County granted the ditch company a franchise to remove obstructions from the old Kaweah River channel and to enlarge it where necessary to a width of 20 feet. Under the franchise work was done on the old river channel to a point near the present head of St. Johns River, about 4½ miles above the present head of the ditch. The river channel was thus kept open until again filled, in places, with drift and silt by the freshets of 1867–68. These freshets washed out the canal head gate in Landers Slough and considerably deepened and enlarged the upper section of the canal. During the period intervening between that winter and 1877, when the making of the Fowler Cut resulted in the practical reestablishment of the old channel of Kaweah River, water reached the head of the Peoples Ditch through a number of high-water channels leading in a southwesterly direction from the upper section of St. Johns River through Visalia Swamp. When water became scarce at the head of the canal, men would be sent up to St. Johns River to increase the flow toward the south by throwing obstructions into that river channel or by opening a better passage through the swamp. There has been a satisfactory supply of water to the head of the Peoples Ditch ever since the Fowler Cut was made.

This cut was made by Mr. Samuel Fowler under contract with the Kaweah Canal and Irrigation Company, which had entered into an agreement with the management of the Peoples Ditch concerning the apportionment of water. The agreement, which was formally executed on December 21, 1878, but subsequently set aside by a decision of the courts, provided for a joint ownership of the upper portion of the Peoples Ditch and of the water way above the head of that ditch as improved by the Kaweah Canal and Irrigation Company, and conceded to the Peoples Ditch the right at all times to as much water as will flow in a ditch 50 feet wide and 2 feet deep on a grade of 6 inches to the mile. The water of Kaweah River which passes the head of St. Johns Channel separates into two channels, each about a mile long, forming an island about 11 miles above Visalia. The upper portion of the

northernmost of these channels is known as the Hamilton or Visalia Cut. The southernmost is Lander Slough, or the enlarged channel of the Lander Ditch. It is the upper portion of Lander Slough which until 1896 carried water to the point where the flow was brought under final control for the Peoples Ditch. But when the canal regulator failed Landers Slough was closed near its head and an artificial canal was substituted for the Landers Slough section of the Peoples Ditch. A box inlet 20 feet wide and 10 feet deep is maintained in the head of this new canal section, which structure seems to be intended more as a partitioner of the flow of Kaweah River than as a regulating gate in the ordinary sense.

The Consolidated Peoples Ditch has a southwesterly course from Landers Slough for 3 miles to the point where its water was separated from that of the Kaweah Canal, so long as that canal was in service. The main canal extends from that point about 1½ miles farther, to a point known as Dillons Mill, where some of its water is in use to supply power. The section of the canal next below Landers Slough follows the original alignment of the Swanson Ditch for about three-fourths of a mile, following and finally crossing the almost obliterated channel of Yokohl Creek. The occasional flow of this creek is admitted into the canal, and surplus waters are discharged through a waste gate into Outside Creek, which was originally a continuation of the Yokohl Creek channel. The main canal is of somewhat irregular dimensions, much of it having the appearance of a natural channel. Its width is generally 30 to 50 feet and its capacity probably exceeds 400 second-feet.

The consolidation of the interests of the several small ditches and individual irrigators, as already explained, led finally to an incorporation of their organization. This was effected in 1874, and the corporation received the name of the Consolidated Peoples Ditch Company. Its capital stock was fixed at $9,700, in 97 shares of $100 each. Canal expenses are assessed upon stockholders. No water is sold by the canal company. At the time of constructing the canal it was agreed that each person interested in the main canal should contribute toward construction down to the point at which his water was to be delivered into a branch ditch. All branch ditches are under independent management. Water is distributed to the various branch ditches, under the direction of the president of the company, in proportion to the number of shares of stock for which water is to be delivered to each. The apportionment of water between the main canal and a branch ditch is accomplished by constructing a sill or floor at the same elevation in each, the effective width of canal and ditch on each sill being made proportional to the amount of water to be passed. The cost of constructing and maintaining these structures falls upon the canal company. The area of land actually irrigated in a single season with water from the Peoples Ditch has exceeded 5,000 acres. The total expendi-

ture on the main canal chargeable to the construction account has been about $50,000.

Swanson Ditch.—This is a branch of the Peoples Ditch, receiving its water about three-quarters of a mile below the point where the latter leaves Landers Slough. It has a southwesterly course and is about 4 miles long. It is reported to have been in use as an independent ditch as early as 1855. While the supply of water from the Peoples Ditch was somewhat unreliable—preceding 1878—irrigators dependent upon the Swanson Ditch endeavored to keep the head of Packwood Creek open, but with indifferent success. Since that time the ditch is to be regarded merely as a branch of the Peoples Ditch.

Lee & Thompson Ditch.—This was a second old ditch, almost parallel with the Swanson for some distance. Its alignment was adopted for the Peoples Ditch for a considerable distance, and only the lower end of the original ditch remains in use as a small distributary of the Peoples Ditch water.

Rice Ditch.—This is another small ditch, now a branch of the Peoples Ditch, which was in use before the latter was constructed. Its course is southerly from a point near the lower end of the main ditch. It is about 6 miles long, terminating in a branch of Outside Creek used as a distributary of the Peoples Ditch water under the name of the Bliss & Hyde Ditch, and it commands a narrow strip of land lying to the east of Outside Creek.

Catron and Teague ditches.—These are two small branches of the Peoples Ditch, both of which receive water from the south side of the latter through a common head about 2 miles below Landers Slough. Their course is southerly. The former is about 3 miles long, the latter about 4, and both are connected at their lower ends with the Rice Ditch, into which surplus water is dropped.

Bliss & Hyde Ditch.—This is a branch of the Peoples Ditch, receiving its water a quarter of a mile below the head of the Catron and the Teague ditches. At that point a short cut from the south side of the Peoples Ditch drops water for this branch into a natural channel—the eastern branch of Outside Creek—which is the same channel used above that point as a common head for the Catron and the Teague ditches. This channel has a course a little west of south for about 12 miles, to the point where the water of the branch ditch is used, about 6 miles east of Tulare, on both sides of Outside Creek, there known in part as Elk Bayou. Water is diverted from Outside Creek into two irrigation ditches, one upon either side, each of which has a length of about 2 miles.

Davis, Brown, and Pennebaker ditches.—These are three short branches of the Peoples Ditch, all receiving water from it at or near the termination of the main canal, and supplying water to lands near Farmersville.

Extension Ditch.—After the use of a portion of the water of the

Peoples Ditch for power at Dillons Mill, which is at the termination of the main canal, it flows into Outside Creek. The creek channel is closed with an overfall dam of brush, which serves as a waste way for all water not required for the Extension Ditch. This ditch is one of the principal branches of the Peoples Ditch. It has a southwesterly course, and, with its branches, commands the region to the west and northward of Farmersville. It was built by the farmers whose lands it waters. They have formed a corporation known as the Extension Ditch Company, whose shares of stock are in the hands of thirty or more irrigators. The affairs of this company are managed entirely independent of the Peoples Ditch Company, which merely delivers to it the water which its stockholders are entitled to receive from the Peoples Ditch.

Kaweah Canal.—The enumeration of this canal is merely historical.

FIG. 5.—Ditch system at head of Kaweah Canal.

It is one of the canals whose successful operation was greatly interfered with by litigation. The canal was constructed in 1877. At that time the Fowler Cut was made for the Kaweah Canal and Irrigation Company, and led to the reestablishment of the original channel of Kaweah River, or of a new channel very nearly coincident with the original one in position. The agreement then made with the Consolidated Peoples Ditch Company gave the new company a right to use the upper sections of the Peoples Ditch for several miles, and below that point a canal was excavated, about 50 feet wide on the bottom and 3 feet deep, which temporarily dropped its water into Deep Creek, in which it flowed to within a few miles of Tulare. The main canal was soon afterwards extended across Deep Creek in a general southwesterly direc-

tion, passing about a mile to the west of Farmersville, until, at a point about 6 miles to the northeast of Tulare, it was separated into two main branches, of which the southerly one commanded the lands in the vicinity of Tulare, including the colony lands toward the west. The northerly branch was extended westerly across Cameron Creek to a connection with the Tulare Canal, and has irrigated a few tracts of land near Tagus. The total length of the main canal, not including the Fowler Cut or the Peoples Ditch, to where it separates into its two principal branches is about 10 miles. Irrigation from this canal has probably exceeded 4,000 acres. The canal was constructed by a corporation known as the Kaweah Canal and Irrigation Company, which was organized in 1877 by farmers who were in need of water for their lands in the vicinity of Tulare. The capital stock was fixed at $50,000, in 500 shares. The expense of canal construction and litigation soon exceeded this sum by at least $10,000. The canal always held its water for sale, but stockholders were entitled to receive it at slightly reduced rates. All expenses in excess of revenue derived from the sale of water were apportioned to stockholders. Water was not sold by measurement; it was delivered to the irrigators from the several canal branches in rotation, and was paid for at fixed rates per acre actually irrigated, generally $1 to $2 per acre. The right of the canal to receive any surplus waters from the Peoples Ditch was disputed in court by farmers who, as riparian owners on Outside Creek, demanded that all flow in the Peoples Ditch in excess of that required for that ditch should be wasted into channels leading into Outside Creek. Their claim having been sustained, the Kaweah Canal for many years received water only by sufferance and only during periods when the river afforded an abundant supply. Its water was no longer taken through the head of the Peoples Ditch, but a cut was made from Kaweah River below Deep Creek to the canal, and the upper half mile of Deep Creek itself was utilized as a source of supply. After the organization of the Tulare irrigation district the canal was sold to the district for $150,000, payment being made in bonds, but with the proviso that the stockholders of the Keweah Canal and Irrigation Company should be preferred takers of one-third of all the water entering the upper section of the canal, at the agreed price of $1.25 per acre per annum.

Farmers Canal.—This canal has its head on the east side of Deep Creek, about 2 miles below Farmersville. It has a southwesterly course, commanding several thousand acres of land between Elk Bayou and Deep Creek channels within 6 miles of the head of the canal. The bed width of the canal for several miles below its head is 14 feet. The head gate or regulator has a width of 18 feet. The water is turned from the creek into the head of the canal by means of a brush dam. The canal was constructed in 1875 by an incorporated company, whose members are the farmers using canal water. The capital stock was

fixed at $20,000, in shares of $100 each. The head of Deep Creek, from which diversion is made for this canal, is about 8 miles, in a direct line, above Visalia, on the south side of Kaweah River. No water flows into Deep Creek from the river during its low stages. Water is generally available from the beginning of March to the end of June. During this time a superintendent is employed to apportion the water to holders of stock. When the supply of water is limited it is used in turn by the irrigators. Each stockholder is permitted to sell or lease the water to which he is entitled. The water represented by a few shares of stock held in the name of the corporation itself is also subject to sale. The estimated cost of canal construction and expenses connected therewith is about $7,000 to $8,000. The annual assessment per share of stock for canal management and maintenance has been about $1. The structures necessary for a delivery of water into branch ditches are constructed and maintained at the expense of those who receive water through each.

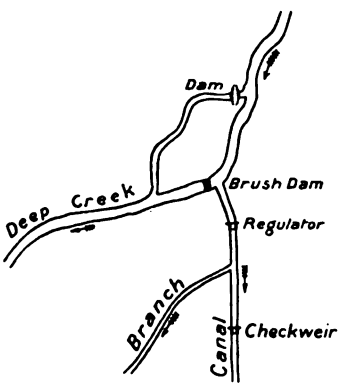


FIG. 6.—Head of Farmers Canal.

Tulare Canal.—This canal was constructed for the irrigation of lands in the vicinity of Tagus and to the northwest of Tulare. Its head is on the south side of Kaweah River, about midway between the head of Deep Creek and the head of Packwood Creek. It has a southwesterly course, is about 15 miles long, and has a bed width of 16 feet. The canal passes about 2½ miles to the south of Visalia and a mile to the south of Tagus, and terminates at the main canal of the Rocky Ford Canal Company. The canal was constructed in 1873. The farmers interested in the enterprise formed a corporation in the following year, known as the Tulare Irrigating Company, whose capital stock was fixed at $40,000, in shares of $100 each. The diversion of water from the river is accomplished by means of a dam or weir of brush, and the inflow into the canal is controlled by a regulator or head gate 16 feet wide. It has been claimed that the Ketchum Cut, one of the several ditches from St. Johns River southwesterly to Kaweah River above the head of the Tulare Canal, has been utilized by the Tulare Irrigating Company to accomplish the diversion of its water direct from St. Johns River, but it has not been possible to verify this claim, or to ascertain whether the canal company has any proprietary interest in the Ketchum Cut.

Before 1885 it was customary to apportion the flow of the canal to the stockholders in the corporation in proportion to their respective interests, but the holders of stock who were not ready to make profitable

LUME OVER TULARE CANAL. THE MAIN CANAL OF TULARE IRRIGATION DISTRICT.

B. MAIN CANAL OF TULARE IRRIGATION DISTRICT.

e of their water demanded a change, and it was at that time decided permit each stockholder to irrigate free of cost as many acres as he d shares of stock, and to charge $1 per acre for all additional areas.

Rocky Ford Ditch.—This ditch, now a part of the Tulare irrigation trict canal system, takes its name from Rocky Ford on St. Johns ver, because half a mile above is the head of the Ketchum Cut, ich was bought by the Rocky Ford Canal and Irrigation Company, l was supposed to deliver water for the canal from St. Johns River o the Kaweah, from which it was reclaimed for the Rocky Ford Ditch the head of Packwood Creek. The head of Packwood Creek, which l been almost obliterated by the freshets of 1861–62 and 1867–68, s reopened by this canal company in 1876. A cut 9 feet wide and ·ee-quarters of a mile long, following the channel of Packwood eek, received the canal water from Kaweah River. The water thus ·erted was carried southwesterly in Packwood Creek, and in its south k about 2½ miles, thence south in a ditch three-quarters of a mile to other natural channel, from which, a half mile below, it was dropped ·o Cameron Creek, which it followed, still in a general southwesterly ·ection, to a point about a mile south of Tagus. At that point the ıter was taken into a system of distributing ditches covering 4,000 5,000 acres of land to the southwest of Tagus, between Cameron and ıckwood creeks. About $20,000 was expended on this work, besides ,000 in litigation, but after a few years the company was deprived of e use of the Packwood Creek channel by an order of court, and those pendent upon this canal found themselves without water.

Cameron Creek Company.—This was an unincorporated organization farmers which may be considered an offspring of the Rocky Ford ıtch Company. The purpose of the organization was to secure water r the Rocky Ford Canal, the head works of which were thrown out of rvice by orders of the court. There was ordinarily a surplus flow water in the spring months, which reached Cameron Creek by way the Deep Creek channels and the Kaweah and Tulare canals. The ıter thus reaching the Rocky Ford Canal from Cameron Creek was stributed to irrigators under the management and control of the ıameron Creek Company. The owners or lessees of stock in the Rocky ɔrd Canal Company were the preferred purchasers of water, but if ere was sufficient available it was also sold to farmers who held no ock. The apportionment of water was by rotation in the use of the ɔw of distributaries. The time of use per share of stock was reduced ; the supply decreased. The charge for water was at the rate of $1 ɜr year per acre irrigated. The necessity for operations by this com- ıny ceased upon the completion of the canal system of the Tulare rigation district.

Bacon & Crossmore Canal (Packwood Creek Canal).—When, in 1875, , became necessary to improve the flow of Packwood Creek for the ıpply of water to a large tract of land 6 to 9 miles west from Tulare,

a ditch was constructed from St. Johns River, then the principal channel of Kaweah River, commencing about a hundred yards above Rocky Ford, thence southwesterly about 4 miles, passing to the north and westward of a group of low hills known as the Swamp Angels, to a connection with the old channel of Kaweah River. Water thus turned into this channel was allowed to flow to a point in Visalia Creek about 1¼ miles below the head of Packwood Creek and was there diverted into a canal about 1,000 feet long leading southerly into Packwood Creek. These two ditches, or canals, together with the natural channel of Packwood Creek, are known as the Bacon & Crossmore Canal. The upper portion of this canal has a width of about 16 feet. The width of its head gate or regulator at St. Johns River is 40 feet. The cut for Visalia Creek into Packwood is about 30 feet wide. Its flow is controlled by a gate 30 feet in width. The point of discharge of the canal from St. Johns River into the Kaweah is just below Goads Dam at the head of the Tulare Canal. To enable diversion of water from Visalia Creek into the channel of Packwood Creek by way of the Bacon & Crossmore Cut, a brush dam is maintained in the creek about 60 feet below the head of the cut. The flow of Packwook Creek as thus established is first checked at a point about 4 miles west of Tagus, where distributaries are in use upon either side of the creek channel. Several other diversions are made some miles farther down the creek, and it is claimed that several thousand acres of land are irrigated by this ditch system.

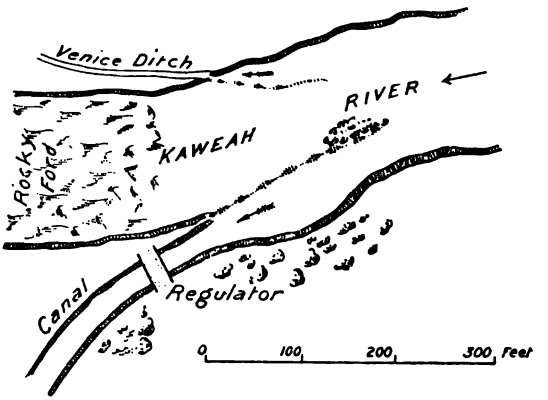


FIG. 7.—Head of Bacon & Crossmore Canal.

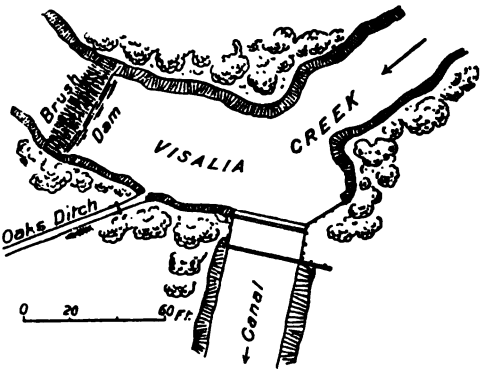


FIG. 8.—Head of Bacon & Crossmore Cut.

Judd Ditch.—This ditch was in use but a short time subsequent to *the freshets of 1867–68.* An organization of interested farmers known

as the Judd Ditch Company at that time endeavored to keep the head of Packwood Creek open, but owing to controversies with people living near Visalia on Visalia Creek the attempt was soon abandoned.

Cutler & Higgins Ditch (*Chatten Ditch*).—This is a small ditch between St. Johns River and Visalia Creek. Its head is on the north side of Kaweah River just above the point where it divides into the two channels, Visalia Creek and Packwood Creek. It has a westerly course, is about 2 miles long, 3 to 6 feet wide on the bottom, and belongs to several farmers, who take turns in using its water for the irrigation of a few hundred acres of alfalfa. The ditch has been in use since 1857.

Kaweah and Mill Creek Water Company.—This corporation was organized in 1877 with a capital stock of $10,000, in 200 shares, for the purpose of harmonizing and protecting conflicting interests by maintaining and controlling the flow of water in Mill Creek (Visalia Creek) and distributing it to the canals and ditches entitled to receive it. Its shares of stock were distributed to the owners of the various canals and ditches according to their relative interests, and the superintendent of this company is required to apportion the amount of water available to each ditch and canal in proportion to the number of shares of stock for which water is to be turned into each. A decision of the courts favoring the riparian doctrine has, however, made it difficult for ditch owners along Mill Creek below Visalia to reclaim their water, as the decision interferes with the construction of dams in the creek channel so long as they are objected to by riparian proprietors. To increase the flow of water in Visalia Creek it was found necessary to open the head of Lane Slough. This had been done before, but later freshets had again obstructed its channel near St. Johns River. The Bostwick Cut was the result of the work done by the company at that point. This is a canal leading southerly from St. Johns River, about a mile above Rocky Ford, into the main channel of Lane Slough. Lane Slough was one of the several natural channels having a southwesterly course through Visalia Swamp from near its northern margin to the old channel of Kaweah River above the head of Visalia Creek. It is claimed that this slough was so obstructed with drift and silt by the freshets of 1861–62 that for reclamation purposes it was necessary once before to open it, under direction of State authorities, with moneys from the swamp-land fund. The water company in 1878 also caused a section of the old channel of Kaweah River just below the head of Landers Slough to be reopened. The work there done resulted in the formation of the Hamilton Cut. All expenses of management incurred by the Kaweah and Mill Creek Water Company are raised by assessments levied upon the stock. The principal ditches dependent upon this organization for their water supply are the Oaks, Watson, Burch, Mill, Bahwell, and Fulgham ditches.

Oaks Ditch.—The head of the Oaks Ditch is on the south side of Visalia Creek, just above the dam maintained below the head of the Bacon & Crossmore Cut. It was in use prior to the organization of

the Kaweah and Mill Creek Water Company, from which it now receives its water. It is a small ditch, flowing several miles westerly between Visalia and Packwood creeks, and supplies water to seven or eight small farms. The entire flow of the ditch is usually divided into two irrigating heads.

Evans Ditch.—This ditch, which has also become dependent on the Kaweah and Mill Creek Water Company for its supply of water, has its head on the south side of Visalia Creek about 2 miles above Visalia. Its upper section serves as a common head for three otherwise independent ditches, the other two being the Burch and the Watson ditches. The Evans Ditch is entitled to one-half of the water entering its head, the Watson to one-third, and the Burch to one-sixth. The ditch has a southwesterly course, is about 5 miles long, and supplies water to about 500 acres just to the south of Visalia. Ownership in the ditch is represented by shares of stock, and the distribution of water is made in proportion to the number of shares of stock owned or controlled by each irrigator. Water is generally available for ten months each year, and is used in turn by the irrigators.

Evans & Turner Ditch.—This is a diversion from the west side of Packwood Creek about 2 miles below the Bacon & Crossmore Cut. It has a westerly course, is about a mile long, and drops its water into the Evans Ditch. It is claimed that this ditch was in use as early as 1858, but if so it was out of service for many years preceding its reestablishment, about 1884.

Burch Ditch.—This is another small ditch to which water is supplied under control of the Kaweah and Mill Creek Water Company. Although the ditch was in use before the organization of that company, and then received water direct from Visalia Creek, it has of late years been supplied with water through the head of the Evans Ditch. Its water, together with that for Watson Ditch, is turned out of the west side of Evans Ditch one-half mile below its head. It flows westerly three-quarters of a mile in a channel used in common with the Watson Ditch, and thence a little south of west to the southwestern portion of Visalia. It supplies a number of small farms southeast of Visalia. The head of the ditch was originally on the north side of the creek and its water was brought to the south side in a flume.

Watson Ditch.—This is another of the old irrigation ditches in whose interest the Kaweah and Mill Creek Water Company was organized. This ditch was constructed in 1854, before water was used by the mill in Visalia. Its head was on the south side of the creek about 3 miles east of Visalia, and its water was used near Visalia. It now receives water through a common head with the Evans and the Burch ditches, and is finally separated from the Burch Ditch water about a mile east of Visalia. Ownership in this ditch is represented by shares of stock, of which 100 may be issued. Between 60 and 70 are in the hands of fifteen to twenty farmers, who use its water mostly on small farms to the south and southwest of Visalia. It was originally intended that

each share of stock should represent 17.28 miner's inches of water (about one-third second-foot), and the original price per share was fixed at $5. Stockholders are assessed for all expenses of management and repairs. A superintendent is employed only when necessary, not when water is abundant, nor when there is none at all. Shares of stock not issued are not entitled to water. Holders of stock may sell or lease the water to which they are entitled. When the flow of the ditch is small it is given to the irrigators in turn for eight hours per share of stock. The annual expense of ditch management is about $5 per share.

Mill Ditch.—This is not an irrigation ditch, but it has rights to the use of water from Kaweah River, which make its enumeration among the Kaweah River ditches desirable. Its head is about one-half mile above Visalia, on the south side of Visalia Creek. It has a westerly course into the town, and drops its water back into the creek at the mill. It has been in use since 1854, at which time a brush dam was constructed at its head in Visalia Creek. This is another of the ditches in whose interest the Kaweah and Mill Creek Water Company was organized. The mill owners are stockholders in this corporation and receive the same consideration accorded to irrigators.

Bakwell and Fulgham ditches.—These are two small ditches below Visalia, which were included among those for whose benefit the Kaweah and Mill Creek Water Company was formed. They and a number of other small ditches make use of the freshet flow of Visalia Creek reaching the lower sections of Mill Creek, as well as of the water which is returned to the creek by the Mill Ditch.

Longs Canal.—This exists in name only. It was a canal or reopened slough constructed, some years after the freshets of 1861–62, southwesterly through the upper end of Visalia Swamp, from St. Johns River to Potter Slough, which discharged into Kaweah River. The first work done there was nominally for reclamation purposes—to facilitate drainage of the swamp. The canal was closed by the drift and silt brought down by the freshets of 1867–68, and was again opened at the expense of the Kaweah and Mill Creek Water Company.

Bostwick Cut.—This is a short ditch or canal, southwesterly from St. Johns River about three-quarters of a mile above Rocky Ford, to Lane Slough, which continues in a southwesterly direction to Kaweah River. It was constructed in 1863 or 1864, for drainage purposes, with money from the swamp-land funds. There seems to be no doubt, however, that it would not have been constructed had it not been desirable to increase the flow in Visalia and Packwood creeks for irrigation purposes. The cut was filled with drift and silt by subsequent freshets, and was again opened in 1876 by the same parties who the following year organized the Kaweah and Mill Creek Water Company. It is about 18 to 20 feet wide on the bottom and half a mile long. It is provided with a head gate having a width of 22 feet.

Ketchum Cut.—This is a ditch, about 8 feet wide on the bottom, having its head on the south side of St. Johns River about a half mile above Rocky Ford. Its course is thence southwesterly, skirting the eastern base of the Swamp Angels to Kaweah River, one-half mile above the head of the Tulare Canal. The ditch was constructed, probably about 1864, with money from the State swamp-land funds, ostensibly for drainage purposes. It was originally carried across Kaweah River. The right to use it for irrigation purposes has been claimed by the Rocky Ford Canal Company, and, possibly, also by the Tulare Irrigation Company. It seems to have been in use principally for the irrigation of few small tracts of land on the north side of Kaweah River. Its flow is controlled by a head gate.

Crowley & Mehrtens Ditch.—To make the enumeration of the old irrigation works complete it is necessary to mention the Crowley & Mehrtens Ditch, which was a small ditch in use for some years on the south side of Kaweah River, westward from Wutchumna Hill. The head of the ditch was near the northwestern base of this hill. There is nothing left of it now except an old cut in rotten granite on the south bank of the river, just below where St. Johns River leaves the Kaweah.

TULARE IRRIGATION DISTRICT.

This district was organized in September, 1889, by landowners in the vicinity of Tulare. Those dependent upon the Rocky Ford Canal and Cameron Creek for their irrigation water seem to have been the prime movers in the matter of district organization. The lands dependent upon Packwood Creek (Bacon & Crossmore Canal) and upon the Kaweah Canal were mostly excluded from the district. The district has an area of 39,200 acres, including the 2,500 acres in the town site of Tulare. After much preliminary work, involving the examination and surveying of a number of feasible canal routes and reservoir sites for water storage, it was decided to purchase the water rights of the Settlers Ditch. This was done for $100,000, in district bonds, through an agent, who at the same time agreed to take an additional block of district bonds amounting to $50,000. The money realized from this bond sale was expended in constructing a canal, 60 feet wide on the bottom, from a point on the north side of St. Johns River about one-half mile below Rocky Ford in a southwesterly direction, nearly parallel with the river, about 2½ miles, to the southern base of Iron Mountain, where the canal water is carried across St. Johns River in a flume 9 feet wide and 4½ feet deep, which is supported by two bridge spans. The course of the canal from St. Johns River is a little west of south to Kaweah River, which is crossed in a similar flume. It then has a course due south to a junction with the Kaweah Canal. Before this entire canal section was completed the available $45,000 had been expended and the district was still without water. The Kaweah and Rocky Ford canals were purchased for $20,000. Contracts were let for the construction of

A. CANAL REGULATOR AT KAWEAH RIVER, TULARE IRRIGATION DISTRICT.

B. LATERAL, ONE MILE SOUTH OF TULARE.

lateral canals and for the enlargement and completion of the main canal, work being practically paid for in bonds which, it was understood, went to the contractor through a nominal purchaser. Main distributaries were carried through the district from northeast to southwest, and smaller ones, generally on land lines from north to south, completed the canal system. The aggregate length of district canals is said to exceed 150 miles, and the main canal capacity is given at 800 second-feet. It should also be said that the completed system of works involves the construction of a storage reservoir at Iron Mountain, to be supplied with water by means of a high line canal with head somewhere near Dillons Point. The reservoir site has in part been purchased. The total bond issue by the district was $500,000, all of which has been disposed of.

The area of land irrigated in the district has been rapidly extended. It is said that from 5,500 to 30,000 acres, principally in alfalfa, cereals, and orchard, have actually been irrigated, the area varying within wide limits, according to amount and duration of the water supply. The inflow of water to the canal is controlled by means of three head gates or regulators, of which the one on the north side of St. Johns River is 60 feet wide, and each of the others 25 feet. They are all of the loose flashboard type, the several openings being about 4 feet, and the drop boards being supported by inclined timbers. A canal superintendent has general charge of water distribution, and he is assisted by ditch tenders. Water is generally available during the four months April to June, inclusive. Operating expenses are met by charging for water, the charge being fixed at 50 cents to $1 per acre for the first irrigation and half price for subsequent wettings. The charge in 1897 was 50 and 25 cents. The tax to meet interest on bonds and cost of its collection has been 1.85 to 1.90 per cent.

WATER SUPPLY, SOILS, AND METHODS OF IRRIGATION.

Prior to the great changes in channel alignment effected by the freshets of 1861–62, the Kaweah River waters spread at high stages over what was known as Visalia Swamp, which commenced at Wutchumna Hill and extended southwesterly about 9 miles, with a width of 1 to 3 miles. The spreading waters were reunited in various channels, as already explained, in and below this swamp. Channel capacity was inadequate to pass flood waters, not only in the swamp, but also below it. It was inexpensive to divert water from a natural channel, or even from a convenient spot in the swamp, into ditches, and the use of water for irrigation commenced soon after the arrival of the first settlers. A number of ditches were thus in use when the regimen of the river was changed by the freshets above mentioned, and entirely new works became necessary in some cases to keep the ditches supplied with water. It was some time before the magnitude of the changes in the river alignment and their ultimate effects were fully realized, and final

relief to those dependent upon the more southerly channels for water did not come until the old channel of Kaweah River was reopened as the Fowler Cut in 1877. Meanwhile there had been a continually increasing demand upon the several river channels for water, and almost every season at the close of the river's high stage there was contention and strife concerning the apportionment of the low-water flow to the different sections of Kaweah delta. Farmers who had settled on Canoe Creek, Elbow Creek, and Cross Creek had become accustomed to regard St. Johns Channel as the principal arm of the river, and dams for its diversion southward into and across Visalia Swamp were not tolerated without protest. When the low-water flow was finally restored to the old channel numerous attempts were made to force it back into St. Johns River, and amicable adjustment seemed out of the question. To avoid the possibility of having a permanent change made at the head of St. Johns River, land was there acquired by the principal south-side ditch companies. A brush dam was then tolerated in the Kaweah River channel, which effected a division of the flow at comparatively low stages, but the exact basis of water apportionment, if any had ever been agreed upon, is not known to the writer. After many years of individual effort some of the settlers were forced into cooperative measures, as already explained, with some degree of success. Had there been greater obstacles to overcome in securing control of water in the first instance, the development of irrigation might have been less rapid, but would probably have been ultimately effected with less friction.

The ditches and canals constructed for the reclamation of Visalia Swamp, as already referred to, were hardly necessary as drain ways, because the increased capacity given the river channels by the freshets of 1861–62 had greatly improved drainage, and they have in a measure become features of the irrigation system of this river.

The surface of the country watered by the many delta channels of Kaweah River has a slope from east to west of about 6 to 7 feet to the mile. The portion of it near Visalia and thence southwest to Tulare, which is generally known as the Four Creek country, is covered, in some places very densely, with a growth of majestic oaks. The soil in the upper portion of Visalia delta is a rich alluvial loam, generally sandy, sometimes heavy. Near the river the soil is deep, resting on sands or gravel. At the base of and between the outlying hills on each side of the river the predominating soil variety is a black loam or dry bog, containing much clay, sometimes gravel, and merges into the heavy red soil of the upper or eastern border of the valley plain. Under the soil of this plain a hardpan layer several feet thick is nearly everywhere found at depths of from 1 to 5 feet. Northward from Kaweah River this valley plain extends in a broad belt along the foothill base. The soil of this belt is of the usual heavy red variety, containing some gravel. Its surface is very generally besprinkled with the low mounds usually known in the West as hog-wallows. The ground water

of this north-side district is from 8 to 30 feet below the surface, depth increasing northward from the river and eastward toward the foothills. Westward from this high east-side plain, and still northward from the river-delta lands, the soil becomes shallower and alkaline salts abound in its surface layers. In the Mill Creek country, 4 or 5 miles westward from Visalia, the hardpan of the higher plain disappears, but the alkaline salts remain present in undesirable quantity.

The soils of the entire delta are generally sandy loams, chocolate colored, deep and strong, without the sublayers of hardpan that are characteristic of the plains. Ground water in the Kaweah (or Visalia) delta is usually at 8 to 12 feet below the surface. Near Tulare and westward from that town the soil is darker than in other portions of the delta. The surface of the soil throughout the delta is smooth and entirely free from the hog-wallows of the adjacent main valley uplands.

Intermediate between the Sierra Nevada foothills and Outside Creek, and southward from Kaweah River, is another belt of country, 3 to 10 miles wide, extending far southward toward Tule River and embracing many acres of excellent, rather heavy soil.

The light average annual rainfall in the Kaweah delta, only about 9 inches, the abundance of natural water courses, and the ease with which water can be diverted into ditches and kept under control, are circumstances which have combined to stimulate ditch and canal construction in this section of the State, but not for speculative purposes. The irrigation works are almost all owned by the farmers who need the water. There are but few irrigators who do not own stock in some ditch company. All irrigation canals in the delta have been carried in the most direct lines from source of water to place of use. They are almost invariably located on lines parallel with the main drainage lines of the country.

The lands watered by Kaweah River are utilized for diversified farming. The natural growth of trees and grasses has always been luxuriant to the limit of the overflow from the delta channels, and the construction of canals and ditches has greatly increased the wet area. Large areas are in use as natural pastures, particularly to the east and north of Visalia. Wheat still remains the staple farm product, but considerable areas have been planted to alfalfa, while the orchard and vineyard and the occasional truck patch give evidence of the rapid movement toward a more intense cultivation of the soil.

The aggregate capacity of the canals, ditches, and water courses whose flow is under the control of ditch companies will probably exceed 2,000 second-feet. There are fortunately many years in which for months at a time, and just when water is most needed, the flow of the river is in excess of this amount. It is difficult to approximate the area irrigated or benefited with water from Kaweah River, because in many instances water is not actually applied to the surface of the land, but reaches the soil indirectly through seepage. There is no way of collecting even imperfect statistics, however, except at great labor in

the field, so that when it is stated that about 50,000 acres are actually irrigated and an additional 40,000 acres are more or less directly benefited with water from this river, too great reliance should not be placed upon the statement.

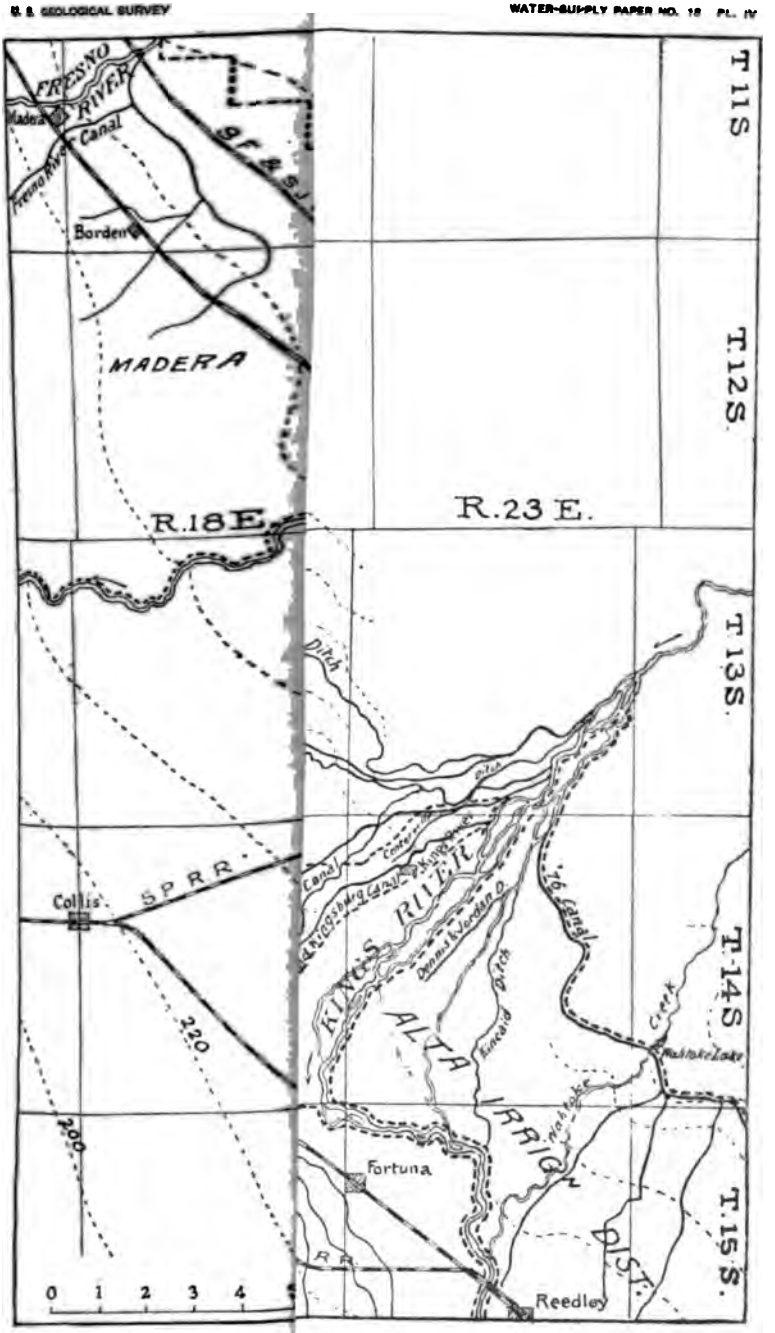
There are very few structures of any kind in the main canals, and but few are required to regulate the grade. Erosion is not undesirable, as the canals are generally long and water is taken from each at comparatively few points; hence the increased cost of providing weirs at points where water is to be diverted is more than offset by the increased capacity resulting from erosion.

All structures for the diversion of water, except dams at the heads of canals, are timber structures; none are of brick or stone. Dams across natural water courses are generally constructed of brush or gravel, and require some annual repairs. No extended observations have been made in this section to determine the loss of water from canals in transit to the lands irrigated; and it is to be remembered in this connection that not all water sinking into the ground from natural or artificial water courses is considered lost, because it is this very loss which fixes the elevation of the plane of subsurface or ground water and the position of this ground-water plane often has a direct bearing on soil productiveness. Throughout a large part of the Visalia district ground water is within reach of the roots of trees, alfalfa, and other plants, being kept there by the water which sinks from natural and artificial channels in transit to lands irrigated and by the water which sinks into the ground on irrigated lands.

The method of applying water to the lands to be irrigated is generally by flooding, in small compartments or checks. These are adapted to the irregularities of the surface, being made rectangular when the surface is sufficiently level and smooth to permit of this arrangement. Crops cultivated in rows, such as beans, potatoes, and corn, also orchards and vineyards, are generally irrigated by running water in small ditches or furrows between rows. In the case of these products the soil is plowed or cultivated as soon after irrigation as practicable.

Here, as elsewhere in San Joaquin Valley, alfalfa is irrigated after each cutting (about every five or six weeks) so long as water is available. Orchards and vineyards receive water frequently during the spring months, and also in the summer if water holds out. Corn is irrigated once or twice, according to the condition of the ground at the time of planting. Grain is rarely irrigated; vegetables, very copiously and frequently.

No attention has as yet been paid to soil drainage. The many comparatively deep natural channels in the delta have kept ground water so far below the surface that the area in need of drainage is relatively small. As irrigation systems are further expanded throughout the delta it may be that subsurface soils will receive water more rapidly than it can pass off from them, and in such case improved drainage systems will become a necessity.


FRESNO RIVER
Madera
Fresno River Canal
S F & S J
Borden
MADERA
T 11 S
T. 12 S.
R. 18 E.
R. 23 E.
T 13 S.
Ditch
Canal
Kings River
Kings River Canal
Denmark Jordan D.
Kincaid Ditch
76 Canal
Creek
Nahtoke Lake
Nahtoke
KINGS RIVER
ALTA IRRIG. DIST.
T. 14 S
T. 15 S.
Collis
S P R R
220
200
Fortuna
R R
Reedley
0 1 2 3 4 5

KINGS RIVER.

HYDROGRAPHY.

The drainage basin of Kings River above the eastern margin of San Joaquin Valley has an area of 1,742 square miles. The river's watershed extends to the main summit of the Sierra Nevada. Less than half of it is to be classed as low mountainous, all the rest being high mountainous country. The snowfall of the winter melts but slowly during the spring months, consequently the river reaches its ordinary high period late in spring or early in summer. The river is generally low from September to the end of January; it is at a medium stage during February, March, and August, and is high from the beginning of April to the close of July. The month of least flow during the six years 1878–1884 was November,[1] the average mean flow having been 313 second-feet, and the least flow for a single month 220 second-feet. The extreme low-water flow of the river has, no doubt, been somewhat below the least monthly mean. At medium stages the river carries 1,000 to 2,000 second-feet of water; during its high stages its monthly mean ranges from 4,000 to over 8,000 second-feet. The maximum flow of the river during the six years above named occurred in June, 1884, the extreme limit having been about 30,000 second-feet.

A gaging station was established on this river September 3, 1895, by the United States Geological Survey. It is located about 15 miles east of Sanger, 3 miles below Jarrett Ranch, and southwest of a hill well named Red Mountain. The station is equipped with a steel cable, car, and tagged wire.[2]

The results of these measurements show for the year 1896 a maximum discharge, which occurred in May, of 22,100 second-feet, and a minimum discharge, occurring in October, of 310 second-feet. The average discharge for that year was 2,582 second-feet. Similar figures for 1897 are as follows: Maximum discharge, 22,730 second-feet, occurring in May; minimum discharge, 250 second-feet, in September, with an average discharge for the year of 2,933 second-feet.

The Southern Pacific Railway Company has maintained gage-height readings at their bridge on the lower Kings River 1 mile from Kingsburg since 1879. The Geological Survey has taken a number of discharge measurements at this point, but owing to the shifting character of the stream it has been found impossible to construct rating tables for same.[3]

[1] See Physical Data and Statistics, Wm. Ham. Hall, State Engineer of California, pp. 452, 476.

[2] Bull. U. S. Geol. Survey No. 140, 1896, p. 286; Eighteenth Ann. Rept., Part IV, 1897, p. 390; Water-Supply and Irrigation Paper No. 11, 1897, p. 92; Water-Supply and Irrigation Paper No. 16, 1898, p. 191.

[3] Twelfth Ann. Rept. U. S. Geol. Survey, Part II, 1891, p. 320, Pl. LXXXII; Bulletin No. 131, 1895, p. 80, Bulletin No. 140, 1896, p. 284; Eighteenth Ann. Rept., Part IV, 1897, p. 393; Water-Supply and Irrigation Paper No. 11, 1897, p. 920, Water-Supply and Irrigation Paper No. 16, 1898, p. 192.

Estimated monthly discharge of Kings River at Red Mountain, California.

[Drainage area, 1,742 square miles.]

Month.	Discharge (in second-feet).			Total (in acre-feet).	Run-off.	
	Maximum.	Minimum.	Mean.		Depth (in inches).	Second per sq mil
1896.						
January	11,020	390	1,474	90,682	0.98	
February	1,140	728	825	47,477	.51	
March	7,020	820	1,710	105,181	1.13	
April	4,600	820	1,938	115,349	1.24	
May	22,100	1,140	5,918	363,890	3.90	
June	18,920	5,160	12,737	757,922	8.15	
July	6,680	1,212	3,742	230,110	2.48	
August	1,212	590	795	48,938	.52	
September	590	390	491	29,234	.31	
October	510	310	350	21,520	.23	
November	1,076	390	538	32,043	.35	
December	550	470	466	28,659	.31	
The year	22,100	310	2,582	1,871,005	20.11	
1897.						
January	624	360	437	26,870	0.29	
February	6,344	1,100	1,631	90,581	.96	
March	4,408	1,240	1,884	115,843	1.22	
April	9,380	1,930	5,318	316,442	3.33	
May	22,732	6,344	14,470	889,731	9.40	
June	10,580	2,520	6,145	365,652	3.87	
July	4,040	1,036	2,177	133,859	1.41	
August	1,100	440	739	45,440	.47	
September	480	250	329	19,577	.20	
October	572	270	394	24,226	.25	
November	2,520	360	692	41,177	.44	
December	8,348	572	985	60,566	.63	
The year	22,732	250	2,933	2,129,964	22.47	

Kings River enters San Joaquin Valley from the east about 20 n east of Fresno. Before reaching Centerville Bottoms the river fl in a broad bed of cobbles from which the hills rise quite abru toward the north and toward the south. The hills retreat from the r somewhat at the upper end of Centerville Bottoms, forming a we shaped valley that opens into the main San Joaquin Valley abo miles farther west, at Centerville. The northern portion of this K *River spur of San Joaquin* Valley may be regarded as a second or w

bottom; the southern portion, lying at a lower level, is the upper division of Centerville Bottoms. These bottom lands, which have a length of about 9 miles, are depressed below the general surface of the San Joaquin Valley plain, being about 10 feet lower than the same at their upper limit and about 60 feet below the same at the Narrows, where the main valley plain drops in abrupt bluffs to the banks of the river as it emerges from these bottoms at their southeastern extremity. From this point southward for about 12 miles the surface of the main valley has greater fall than the river, gradually approaching its level, and near Kingsburg the bluffs upon either side of the river entirely disappear. It is here that the Kings River delta begins.

Through Centerville Bottoms the river flows in numerous channels,

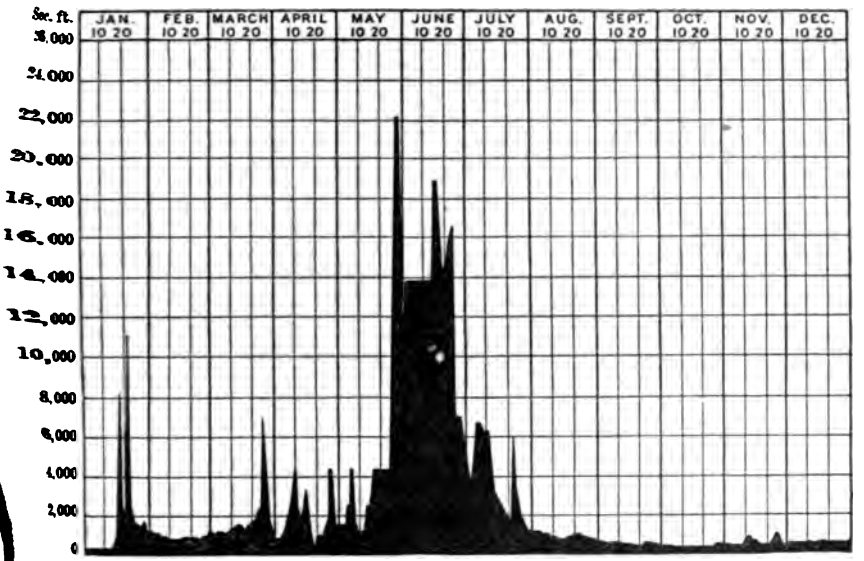


FIG. 9.—Discharge of Kings River at Red Mountain, California, 1896.

the main stream having a length of about 14 miles and a total fall in that distance of about 120 feet. The beds of these streams are for the most part formed of deposits of cobbles, gravel, and sand. Channel banks are rarely high or well defined. Below the Narrows the fall of the river is only 2 feet to the mile, which is maintained with reasonable uniformity to the outfall of the main channel into Tulare Lake.

From the head of Cole Slough, 2 miles south from Kingsburg, the main channel of Kings River has a southwesterly course for a distance of 35 miles to the trough of San Joaquin Valley; its course thence is south about 11 miles to Tulare Lake.

Cole Slough, the uppermost arm of the river thrown off toward the north, has a course nearly parallel with the main channel of the river for some miles, then flows northwesterly and, together with other delta

channels, delivers its water into Fresno Swamp, through which it flows into San Joaquin River. Kings River has but a single tributary below the point where it enters San Joaquin Valley. This is Wahtoke Creek, a small foothill stream which reaches the river from the east at the Narrows. Owing to scant rainfall, there is but little run-off from the lands of the plains, and this is, as a rule, away from the main stream and not toward it.

Canals and ditches from this river are to be divided into two principal groups, of which one embraces those which leave the river at the upper end of Centerville Bottoms and the other those of the delta. For

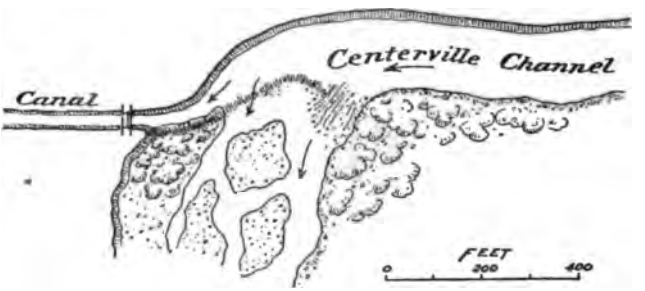


FIG. 10.—Head of Kings River & Fresno Canal.

convenience these groups may be again subdivided according to location of tracts irrigated, and will be treated under appropriate subheadings.

CANALS AND DITCHES.

Kings River & Fresno Canal.—This is the upper north-side canal from Kings River. Its head is 5½ miles above Centerville. The canal receives water through the head of a natural channel or arm of the river which has a position at the base of the low north-side slope that rises from the border of the river bottoms. Within a mile of its head the canal is carried in a cut northwestward out upon the surface of the second bottom, and a half mile beyond its water is dropped into a small natural channel known as Burns Slough. After flowing in this southwesterly for a mile, the water is carried westward in a cut less than one-half mile in length to Kip Slough, from which it is again reclaimed 1½ miles below. The course of the canal, now in an artificial channel, is thence westerly across Fanshaw, Red Bank, and Frolic creeks into the Dry Creek district northeastward from Fresno.

The cut from Centerville Channel of Kings River to Burns Slough has a bed width of about 30 feet. Below that point the canal width is quite irregular, ranging from about 15 to 24 feet. The flumes over creeks are 16 feet wide, with sides 3 feet high. The longest one is that across Red Bank Creek, which has a length of 96 feet. The fall of the main canal is far in excess of what might be considered necessary, its course being almost in the direction of greatest fall of the ground's

KINGS RIVER AT UPPER END OF CENTERVILLE BOTTOMS, LOOKING UPSTREAM.

surface. This results in no injury to bank lands in the upper section of the canal, where a firm substratum sets a limit to erosion, but has made the use of frequent check weirs or drops necessary in its lower sections, particularly at points where water is delivered into branches.

The construction of this canal was commenced in 1872. At that time a regulator was placed in its head at Centerville Channel of Kings River. The following year the cut to Burns Slough was made and the canal was extended westward upon the valley plain. The canal was intended primarily to supply water to landowners who had become stockholders in it, but the sale of water was not restricted to the stockholders. The company's capital stock was fixed at $100,000, in 1,000 shares. Water is sold at rates annually determined, not exceeding $50 per second-foot for stockholders and $75 per second-foot for other irrigators. The sale of water covers all expenses of management and repairs. The first cost of the canal was about $50,000.

As a result of adverse claims to the use of Centerville Channel the canal company was in conflict with the Fresno Canal and Irrigation Company until, in 1875, it was agreed that the water in the regulator of the Kings River & Fresno Canal should be maintained at 1 foot in depth during the low-water period of the river until there was less than 1 foot in depth in the regulator of the other canal, and that thereafter equal depths should be maintained in both regulators.

The distribution of water to ditches of consumers is in charge of a canal superintendent. No system of measurement is in use. Water is delivered to each ditch on the judgment of the superintendent in proportion to the number of second-feet to which each ditch is entitled. No attempt is made to effect actual delivery of the amount of water bargained for. Any number of irrigators may unite and have the aggregate amount to which they are entitled delivered through one gate.

The Enterprise Ditch was the result of such combination of interests. It was constructed about 1885. It received water for a time from the main canal at the point where its waters are dropped into Kip Slough, and takes a westerly course along the base of the north-side foothills. Rounding the point of these about 2½ miles north of Centerville, its course is northerly for 4 miles, thence westerly about 8 miles, generally 3 to 4 miles north of the main canal. This ditch was, a few years after construction, sold to the Fresno Canal and Irrigation Company in consideration of 40 water rights that were issued to its owners.

Fresno Canal.—The head of Fresno Canal is about 1½ miles below the head of the Kings River & Fresno Canal; also on the north side of Centerville Channel. A short artificial channel having a westerly course drops the water of the canal into Chambers Slough, which is a depression at the base of the low bluff at the northern margin of the bottoms. This has been converted into a section of the canal by the construction of a levee along its south bank to a point a little over a

mile from the head of the canal. There the canal enters Long Cut, which extends westward in a direct line a little over one-half mile and carries the canal water into another natural channel, Burns Slough, which connects with Kip Slough a short distance below. Long or Big Cut pierces the rimland which borders Centerville Bottoms on the north. It has a greatest depth of about 10 feet and a width of 65 feet. Kip Slough is utilized for the canal for a little over a half mile. It is thence carried westerly nearly 2 miles in an irregular natural depression, which was converted into a canal by constructing embankments along the south bank of the proposed water way. From this depression an artificial channel only about 500 feet long drops the canal water

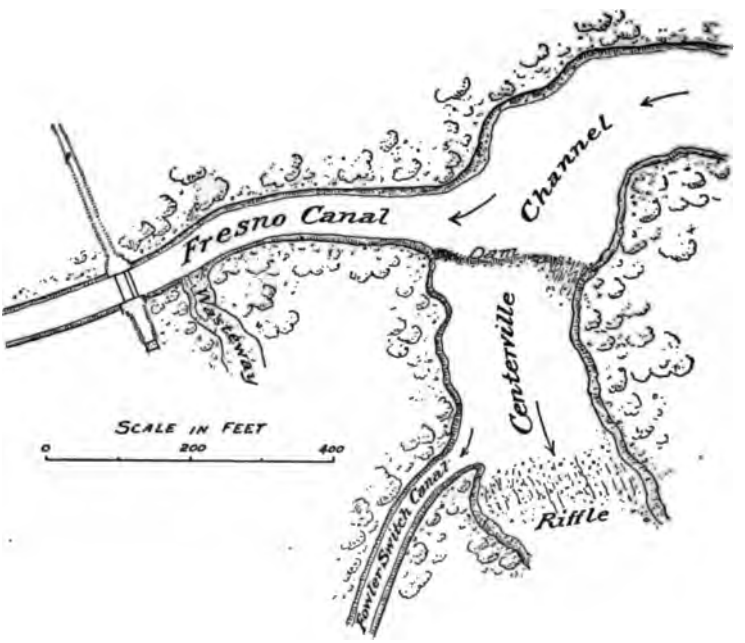


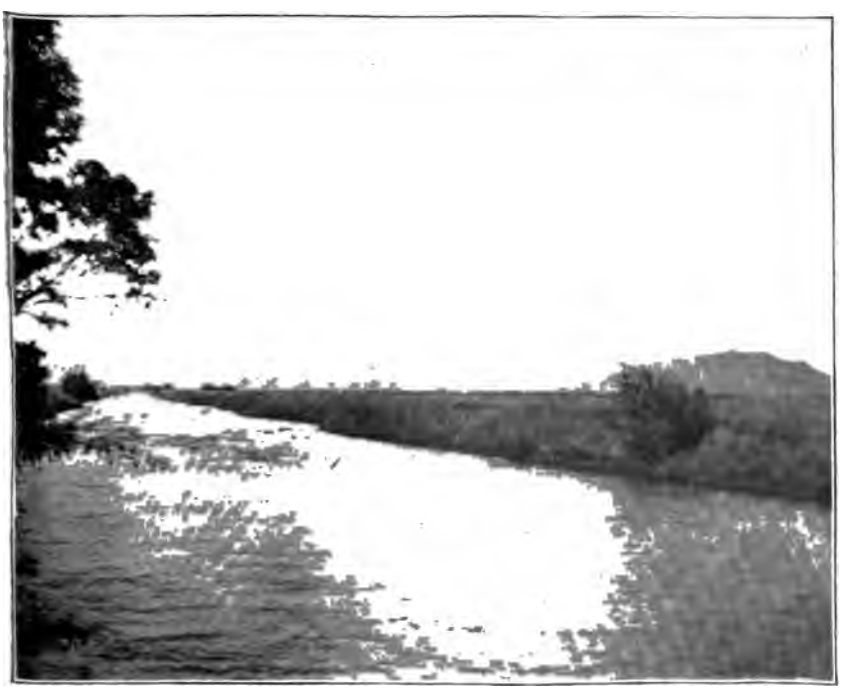
FIG. 11.—Head of Fresno Canal.

into Mud Creek, down which it flows in a southwesterly direction 3 to 4 miles to Fancher (Fanshaw) Creek, which then becomes the main channel of the creek to a point about 3 miles east of Fresno.

Where Fresno Canal leaves Kip or Lone Tree Slough the slough is closed by means of an embankment, in which a gate has been placed, through which water is delivered into the lower section of the slough, which is also known as the Kingsburg branch of Fresno Canal. This branch delivers water to the Highland, the Garfield (which also receives water from other sources), the McCall, and the Selma ditches.

The main canal may be regarded as terminating in the old sink of Fancher Creek, about 3 miles east of Fresno. From that point it sends southward its Washington Colony branch and southwestward

A. REGULATOR ON KINGS RIVER AND FRESNO CANAL



B. FOWLER SWITCH CANAL NEAR SANGER, LOOKING UPSTREAM

Central Colony branch, from which there is another branch to sno Colony. At the Hobler Dam, about 8 miles east of Fresno, a nch is taken out toward the south and reaches Fowler and the col-es in its vicinity, and another branch is carried westward from the th side of the main canal to portions of the Easterby ranches. o miles above the Hobler Dam the principal north-side diversion is de. This is known as the Mill Ditch, which for many years furnished ver to a flour mill in Fresno, and delivered the water, after use for ver, to the colonies southwestward from Fresno. The ditch through sno and this north-side branch has become the feeder of the Bank nd Canal, which conveys water northwesterly to a large tract of d northwest of Fresno. Another north-side branch of the Fresno nal is the Enterprise Ditch, which has already been referred to in cribing the Kings River & Fresno Canal (p. 43). This ditch was chased for 40 water rights.

To attempt will here be made to describe or even to enumerate the ny small branches of this canal, particularly as some of these are ned independently and often receive water from other sources as well from the Fresno Canal. It may be sufficient to state that the aggre-e area covered by water rights (each nominally 1 second-foot to 160 es) exceeds 160,000 acres.

The history of this canal construction is not without interest. Oper-ons on it commenced in 1870 with the posting of a notice of a claim water "to be taken from Kings River at the upper end of Sweem's tch, 20 feet on the bottom, 30 feet on the top, 4 feet deep." This im was made by Mr. M. J. Church, who a month later acquired a o-thirds interest in the Sweem Ditch. The Sweem Ditch was a small tch, the construction of which seems to have been commenced in 1870, hich was intended to increase the flow of what was then known as e Centerville Ditch. Its upper section was enlarged and became the ad section of Fresno Canal. In 1872 a regulator or head gate was nstructed in the head of the Fresno Canal, and in the spring of 1874 small cut on the line of the present Long Cut was completed from the nterville Ditch to the lower sections of the Fresno Canal, on which ork had meanwhile been pushed to the limit of the means of the pro-tor of the work, Mr. M. J. Church. This small connecting ditch is ported to have had a bed width of only 5 feet on a grade of 5 feet to e mile. As the alignment of a part of the older Centerville Ditch s in time to become the alignment of a section of the Fresno Canal, d because it seemed desirable to secure the water rights acquired by e Centerville people, arrangements were made to obtain control of the rporation known as the Centerville Canal and Irrigation Company, and ransfer of their ditch property was made to the Fresno Canal and Irri-tion Company. This was in 1874. As soon, however, as the settlers r Centerville realized what their officers were doing the demands for t treatment at the hands of the Fresno Canal and Irrigation Com-

pany were made so emphatic that those who were entitled to water in the older ditch were granted water rights in the new canal for their lands in perpetuity, without limit as to quantity and free from expense assessments. About 1,500 to 2,000 acres of land are thus covered. In 1875 it became necessary to enlarge the cut westward from the Centerville Ditch, and to this end labor was bargained for with farmers who wanted water delivered to their lands through Lone Tree Creek. Thirty-five water rights were issued to farmers, who completed it in June, 1875. Meanwhile other parties besides Mr. Church had become financially interested in the enterprise, notably Mr. P. Y. Easterby, and later Mr. W. S. Chapman. It was not profitable, however, at the outset, and, as the indirect result of overspeculation in wheat by outside parties and failure to increase loans, a transfer of the property in 1876 to the Bank of Nevada became necessary. The management of the property by this bank was expensive and gave no promise of success. It was not long, therefore, before the original projector of the enterprise was sent for and the property was sold to him, on the execution by him of a long-term note, for $28,000. By this time a number of large holdings of land had been subdivided and sold in small tracts and had become dependent upon the canal for water. Notable among these early colonies are Central Colony, 2,640 acres, and Temperance, Church, and Nevada colonies, each 640 acres. These colonies, together with the Pioneer vineyard on the F. T. Eisen tract, soon demonstrated the great productiveness of the Fresno sand plains, and the development of that region was rapid during the next decade. The sale of water rights had progressed steadily, so that in 1884 about 400 had been issued or bargained for, and their price had increased from $200 to $800 each. Later their price was fixed at $1,600 by the canal management.

A water right, as issued by the company, is the right to use one-thousandth part of the flow of the canal, not in excess of 1 second-foot, on a specified tract of 160 acres. The canal company reserves the privilege of issuing 1,000 water rights without enlarging the main canal. Each purchaser of a water right agrees not to use the water, or permit it to be used, on any other land than that for which the right is purchased, nor to permit the water to run off upon contiguous land or in any other way to run to useless waste, and he agrees to return surplus waters back into the main canal or a branch thereof. Each water right remains subject to an annual expense assessment, which ranges from $80 to $100. Water is delivered without any attempt at measurement to each irrigator at any point on the company's ditch system that he may select. No transfer of a water right can be made except with the land which it covers. Each purchaser of a water right grants to the canal company the right of way for ditch and canal purposes through any lands lying in the same township as the tract for which water is bought, and concedes to the company the right to use his private ditch provided the company does not use it so as to

COBBLESTONE AND BRUSH DAM OF FRESNO CANAL, KINGS RIVER.

interfere with the delivery of his water. The purchaser of a water right further obligates himself to pay to the canal company annually an agreed sum (this has usually been $100), and in default of payment for thirty days to forfeit his water right. The water-right agreement further sets forth that the company shall not be responsible for deficiency in water supply caused by drought, insufficiency of water in the river, hostile diversion or obstruction, forcible measures or temporary damage by floods or other accidents, but that it shall use and employ all due diligence at all times in restoring and protecting the flow of water in its canals.

The structures on the canal do not deserve extended notice. They are all made of timber. The original regulator has been replaced by a second one, constructed about 1884, shown in Pl. VIII, *A* and *B*. It consists of a substantial framework of heavy timbers, spaces between vertical posts being closed by vertical sliding gates. From the head gate an embankment of cobblestones, gravel, and sand extends northward across Centerville Bottoms to a connection with high land, and prevents destruction of the upper section of the canal during freshets. The diversion of water into the head of the canal is effected by means of a brush-and-cobblestone dam, as shown in Pl. VII, the maintenance of which in the past has not been expensive. Along the line of the canal, wherever necessary to prevent excessive erosion, or where required to turn water into a branch ditch, light timber weirs are in use. Most of these consist of two side or bulkhead walls and a floor, usually placed a little below the grade of the canal, and a line of vertical posts supported by braces from below. Spaces between posts are closed to the desired height either by fixed horizontal boarding or by movable drop timbers or boards.

The impression which the whole work gives is that of haphazard management, the result of which is not, however, entirely unsatisfactory.

The control of the canal long ago passed out of the hands of Mr. Church, who effected a profitable sale of the property. It is now managed by parties who also own controlling interests in the Kings River and Fresno Canal Company and who have acquired the property known as Rancho Laguna de Tache. It remains to be added that these parties as owners of the rancho, which is located on lower Kings River, have entered into a contract with the Sunset irrigation district under the terms of which, in consideration of a reservoir site, a right of way for a canal, and a waiver of riparian rights, the irrigation district has paid to the owners of the rancho $250,000 in bonds and has agreed to make no demand for water until 3,500 second-feet of water are available for the Fresno Canal and the canal of the rancho.

Centerville Ditch.—This is at present a branch of the Fresno Canal. It was constructed in 1868 and 1869 by settlers near Centerville (now Kings River). They organized a company known as the Centerville

Canal and Irrigation Company, which was incorporated in August 1868. This company seems to have been the successor in interest to all rights acquired by some of the settlers whose first steps to secure water were taken in 1865. The canal was constructed southwesterly from a point near where the head of the Kings River & Fresno Canal now is about 2½ miles to Burns Slough; thence this slough was utilized for about a mile to a point a short distance below the Long Cut, and thence the ditch was extended southwesterly about 3 miles farther to lands at and west of Centerville. The canal property was transferred to the Fresno Canal and Irrigation Company in 1874, by those who had secured control of the stock, and the opposing minority interests were finally placated by the issuance of perpetual, unassessable water rights in the newer canal to stockholders who owned lands near Centerville.

Sweem Ditch.—This ditch was never completed. Its construction was commenced in 1870 from a point near the present head of the Fresno Canal, practically on the alignment subsequently adopted for that canal. Its purpose was to increase the delivery of water into Burns Slough for the Centerville Ditch. It was sold before completion to the parties who were at work on the Fresno Canal project, and within a few years it was transformed into a large canal, as already explained.

Fowler Switch Canal.—The head of the Fowler Switch Canal is in Centerville Bottoms, about 100 yards below the head of the Fresno Canal. Its course is southwesterly for 1½ miles in Centerville Bottoms, thence westerly for 2 miles across the second bottoms to near the channel of Lone Tree Creek, thence southwesterly 10 miles and southerly 5 miles to a point about midway between Fowler and Selma, thence southwesterly 5 miles, thence northwesterly into the district southward from the Washington Colony. The canal is said to have been built to carry 1,500 second-feet. Its bed width is about 45 feet, its depth variable. The gradient is very irregular, being that of the natural surface of the ground in its upper sections, where a quite firm hardpan formation is relied upon to check excessive erosion of the canal bed. Near the lower end of the canal it was given a fall of 1.92 feet to the mile.

It was proposed to use no check weirs to reduce the fall, which for short distances was as great as 12 feet to the mile, and to put lateral ditches at a lower elevation than the main canal, so as to reduce the necessary structures to a minimum. It was soon found, however, that weirs would be necessary at some points to check erosion and at most points where water was to be diverted.

Among the branches of the Fowler Switch Canal may be mentioned the Cleveland Ditch, about 5 miles long, which delivers water to lands midway between Malaga and Fowler; the Western Canal, which supplies water to lands adjacent to and immediately south of the Sierra

A. REGULATOR OF FRESNO CANAL ON KINGS RIVER, AS SEEN FROM ABOVE.

B. REGULATOR OF FRESNO CANAL ON KINGS RIVER AS SEEN FROM BELOW.

Park and Washington Colony; the Grant Ditch, which has a westerly course for 4 or 5 miles from near the end of the main canal; and the Elkhorn Canal, which has a southwesterly course and delivers water to the western portion of the Wildflower region. The entire canal system gave promise of being one of considerable importance to the prosperity of the district it commanded, but hopes have not been fully realized. The canal has not been permitted to take river water without protest, and unfavorable court decisions have thrown some doubt upon the reliability of this canal as a source of supply.

This canal was constructed in 1883. Farmers in the vicinity of Fowler who wanted water for their own lands formed a corporation,

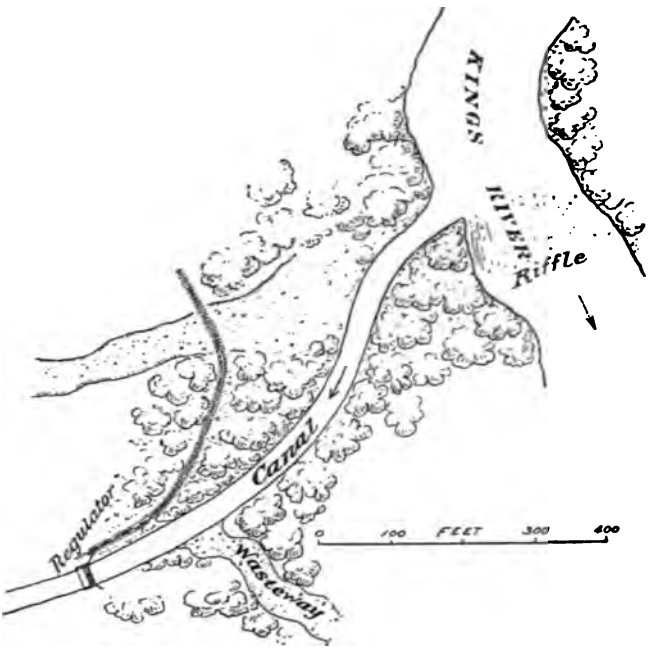


FIG. 12.—Head of Fowler Switch Canal.

the capital stock of which was fixed at $450,000, divided into 1,500 shares, of which 300 were subscribed for. It was agreed that in payment for each share labor and material might be contributed to the extent of $200, the remainder to be paid in coin. It was soon found, however, that the first issue of shares would not cover the cost of canal construction, and it was followed by a second issue of 300 shares, which were taken by the same persons who held the first issue. About $110,000 were expended on canal construction in the first two years.

The water in the canal is apportioned to the stockholders, who may ask for the delivery of their water at any point on the main canal. This has led to the construction of a number of branch ditches under

independent management, which receive their water from the main canal. One of the principal branches of this sort is the Elkhorn Ditch. It is owned by a company in which thirty shares are represented. Some of the stockholders in the Elkhorn Ditch Company are not holders of Fowler Switch stock. In such cases they are required to buy their water from some stockholder. The use of the Elkhorn Ditch is restricted to its owners, no water being sold.

The water represented by the stock of the Fowler Switch Canal Company is at the disposal of the individual stockholders to the extent that it may be leased or sold and its delivery may be called for through any branch ditch. No system of water measurement or special design of gate has been prescribed. All gates in distributing ditches are constructed by the consumers. The delivery of water through these gates is in charge of a canal superintendent. The canal company itself does not sell any water. All expenses are met by assessing the stockholders.

The structures on the canal do not merit extended notice. They are all of very light construction. The check weirs or drops are V-shaped, upper and lower faces being permanently sheathed with light boarding. There is no diverting dam at the head of the canal, which has been located just above a natural cobblestone-and-bowlder riffle.

Centerville & Kingsburg Canal.—This is another of the important north-side canals from Kings River belonging to the upper group of canals. Its head is about 2 miles in a direct line northeast of Centerville, and about the same distance by river below the head of the Fowler Switch Canal. Its course is westerly for nearly half a mile, directly toward the northwestern margin of Centerville Bottoms. Its second half mile has a southwesterly direction, along the slope from higher plains to the bottoms; thence on the higher level it holds a course southwesterly and southerly for about 8 miles, practically parallel with the bluff, 20 to 30 feet high, which drops from the valley plain to Centerville Bottoms. This portion of the canal is frequently within a few rods of the edge of the bluff, and is rarely more than half a mile distant therefrom.

About 9 miles below its head the canal is separated into a number of branches, which radiate throughout the district westward from Kings River below the Narrows, extending southeastward to lands east and south of Sanders, southward to the vicinity of Kingsburg, and southwestward beyond Selma and well into the Wildflower country.

The main canal has irregular dimensions, its bed width being generally about 30 to 35 feet. It has been allowed to cut deep into the friable surface soils in the upper sections of its course, where but little attempt has been made to reduce its gradient below that of the natural surface of the ground. The length of main canal is about 18 miles. The length of its three principal branches is reported at about 26 miles.

The diversion from the river is effected by means of a brush-and-cobblestone dam, in which a small gate serves as waste way and sand

A. REGULATOR ON FOWLER SWITCH CANAL AS SEEN FROM ABOVE.

B. REGULATOR ON FOWLER SWITCH CANAL AS SEEN FROM BELOW.

independent management, which receive their water from the mai canal. One of the principal branches of this sort is the Elkhor Ditch. It is owned by a company in which thirty shares are represented Some of the stockholders in the Elkhorn Ditch Company are not hold ers of Fowler Switch stock. In such cases they are required to bu their water from some stockholder. The use of the Elkhorn Ditch i restricted to its owners, no water being sold.

The water represented by the stock of the Fowler Switch Canal Com pany is at the disposal of the individual stockholders to the exten that it may be leased or sold and its delivery may be called for throug any branch ditch. No system of water measurement or special desig of gate has been prescribed. All gates in distributing ditches are co structed by the consumers. The delivery of water through these gate is in charge of a canal superintendent. The canal company itself doe not sell any water. All expenses are met by assessing the stockholder

The structures on the canal do not merit extended notice. They a all of very light construction. The check weirs or drops are V-shape upper and lower faces being permanently sheathed with light boardin There is no diverting dam at the head of the canal, which has be located just above a natural cobblestone-and-bowlder riffle.

Centerville & Kingsburg Canal.—This is another of the importa north-side canals from Kings River belonging to the upper group canals. Its head is about 2 miles in a direct line northeast of Cent ville, and about the same distance by river below the head of t Fowler Switch Canal. Its course is westerly for nearly half a mil directly toward the northwestern margin of Centerville Bottoms. It second half mile has a southwesterly direction, along the slope from higher plains to the bottoms; thence on the higher level it holds a course southwesterly and southerly for about 8 miles, practically parallel with the bluff, 20 to 30 feet high, which drops from the valley plain to Centerville Bottoms. This portion of the canal is frequently within a few rods of the edge of the bluff, and is rarely more than half a mil distant therefrom.

About 9 miles below its head the canal is separated into a number o branches, which radiate throughout the district westward from King River below the Narrows, extending southeastward to lands east an south of Sanders, southward to the vicinity of Kingsburg, and south westward beyond Selma and well into the Wildflower country.

The main canal has irregular dimensions, its bed width being gene ally about 30 to 35 feet. It has been allowed to cut deep into the friabl surface soils in the upper sections of its course, where but little attemp has been made to reduce its gradient below that of the natural surfac of the ground. The length of main canal is about 18 miles. The lengt of its three principal branches is reported at about 26 miles.

The diversion from the river is effected by means of a brush-and cobblestone dam, in which a small gate serves as waste way and san

A. REGULATOR ON FOWLER SWITCH CANAL, AS SEEN FROM ABOVE.

B. REGULATOR ON FOWLER SWITCH CANAL, AS SEEN FROM BELOW.

sluice. The canal regulator is close to the river and is of a type very common on Kings River canals. Three rows of posts rising from a plank floor extend across the canal from one bulkhead wall to the other. They support a platform loaded with cobbles and gravel to give stability to the structure. The spaces between upper posts are closed by means of vertically sliding gates, each of which has a stem extending above the platform, to which power is applied when the gate is to be opened or closed. The cost of the regulator was about $1,000. The first cost of the canal was about $35,000.

This canal was constructed in 1877 and 1878. It was built by a company organized by farmers who owned lands near Kingsburg and Selma. The capital stock of the corporation which they formed was originally fixed at $12,500, but was subsequently increased to $35,000, being divided into fifty shares. The upper 2 miles of the canal were constructed by day labor. The rest of the main canal was divided into

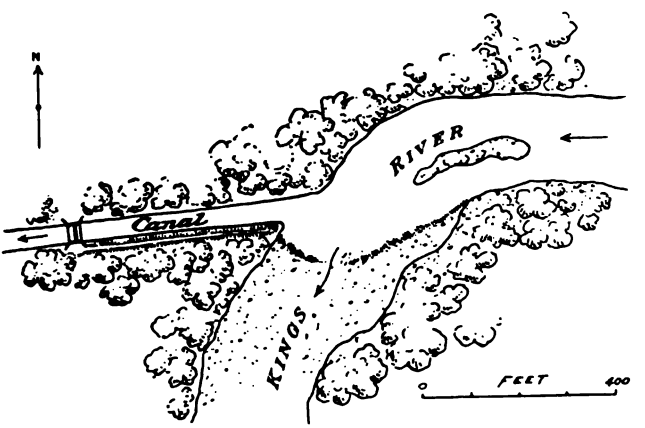


FIG. 13.—Head of Centerville & Kingsburg Canal.

sections, each of which represented one share of stock, and was assigned to some stockholder, who was required to complete it. A few unlimited, unassessable rights to water from the canal have been granted to settlers near Centerville, in consideration of rights of way. Each stockholder is entitled to the delivery of a proportional part of the water remaining available for distribution; but each stockholder, or the corporation when in possession of any shares of stock, may lease the shares. The price of a share per month in 1882 was $6. Shares were at that time valued at about $1,500. Each share of stock is supposed to represent "3 feet of water;" an expression of quantity which seems as vague as the amount named in the franchise of the company—"150 cubic feet of water under a 4-inch pressure." In fact, however, each share of stock entitles its holder to a proportional part of the water varying somewhat according to the number of shares in actual use, but not in excess of three times the amount which would flow 4 inches deep

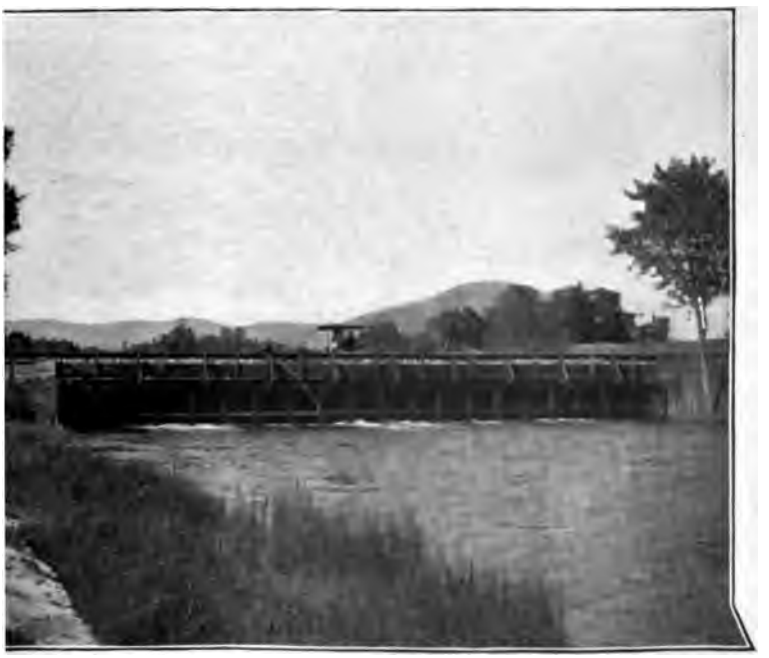
over a clear overfall 12 inches long. Water is apportioned on the judgment of a canal superintendent, generally through gates of like construction. When the amount of water per share of stock in the canal exceeds three units, each indicated by a clear overfall of 4 inches on a length of 12 inches, then all surplus water is held for sale, by the directors of the company, at fixed rates per month per unit. The cost of canal management, maintenance, and repairs is assessed upon the stockholders. The annual expense assessment is now about $80 per share of stock. The works required to divert water from the main canal into private ditches, including weirs in the main canal, are constructed by the consumers of water, subject to approval of the canal company.

Satisfactory use of this canal has been interfered with to no little extent by litigation, generally involving the right of the canal to receive water from the river, and, as in the case of other canals from Kings River, its permanent efficiency as a source of supply has been thrown somewhat into doubt by adverse decisions of the courts.

Selma irrigation district.—This irrigation district is situated on the north side of Kings River, in Fresno County, and embraces within its limits the lands now watered by the Fowler Switch and the Centerville & Kingsburg canals. It was organized in 1890 with an area of 271,000 acres. For the purchase of existing canals and the construction of new works, it was proposed to issue bonds to the amount of $1,000,000; but the bond issue was twice defeated at the polls, and it seems to be generally conceded that there is small probability of a continuation of operations under the district law.

Alta irrigation district (76 Canal).—The canal which supplies water to the Alta irrigation district is known as the 76 Canal. It belongs to the upper group of Kings River canals. Its head is on the south side of the river, about 1½ miles above the head of the Kings River & Fresno Canal, if the point at which the uppermost work for the direction of water toward the canal regulator may be termed its head. At that point a branch of the river has been enlarged somewhat and otherwise improved, and the water diverted from the main stream through this natural channel is carried in a succession of depressions or high-water channels along the southeastern margin of Centerville Bottoms about 3 miles, to the point where the canal cuts out upon the higher plain lying to the east of Centerville Bottoms. The natural depressions of the upper coal section have in large part been converted into a canal by building up, along their low western banks, embankments of cobblestones and bowlders, which were taken from their beds for their enlargement. Where the canal turns away from Centerville Bottoms a cut 9 feet in depth was necessary. From this point the canal has a general southeasterly course, following a grade line with 18 inches fall to the mile, that skirts the western foothill base. At 6 miles below the regulator the canal is carried across Wahtoke Creek, and its southeasterly

A. REGULATOR ON SEVENTY-SIX CANAL, AS SEEN FROM ABOVE.



B. *REGULATOR ON SEVENTY-SIX* CANAL, AS SEEN FROM BELOW.

course is maintained about 22 miles farther, to near Cottonwood Creek. The main canal has been made the eastern boundary of the Alta irrigation district.

The bed width of the canal throughout the first 9½ miles below the regulator is 100 feet. Its capacity, with a depth of 5 feet of water, is about 1,200 second-feet. From the main canal numerous branches extend southwesterly and southerly into the heart of the district. The principal of these is the Traver branch, which has been given a bed width of 60 feet. It leaves the main canal about 1½ miles below Wahtoke Creek, and, as its name indicates, delivers water to lands in the vicinity of Traver. Branch canals are for the most part located across the valley plain on lines of its greatest slope. They have generally been given positions on high ground intermediate between gentle depressions, though in some instances, as in the case of the Traver

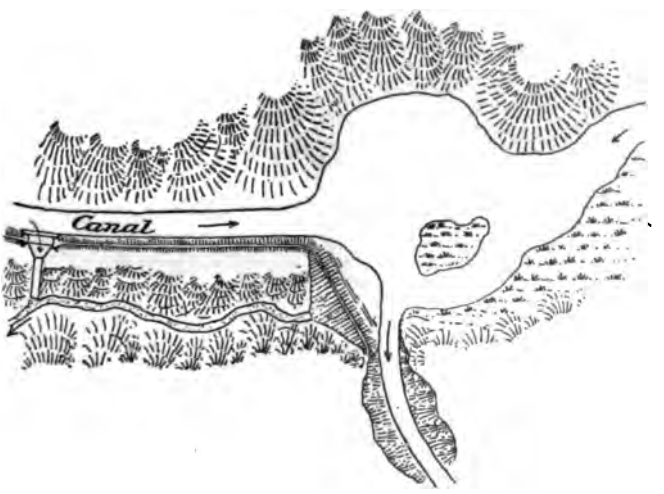


FIG. 14.—Wahtoke Reservoir, on 76 C nal.

branch, natural water courses have been in large part substituted for expensive canal work.

The inflow of water from the upper canal section into the second section of the main canal is controlled by means of a regulator or head gate. Above the regulator in the west canal bank is a spillway through which surplus waters are dropped into Patterson Slough, one of the Kings River channels in Centerville Bottoms. The regulator rises from a plank floor constructed of two layers of 3-inch planking, the dimensions of which are 100 feet in the direction across the canal by 30 feet along its axis. The surface of the floor is at grade of the canal bottom. Along its upper and lower edges are two lines of sheet piling, not driven, but constructed in trenches, which extend to a depth of 5 feet. The bottom ends of the sheeting planks are nailed to a horizontal plank 3 by 12 inches, which adds to the effectiveness of the sheeting. Trenches were

refilled with carefully selected material well tamped into place. Thr rows of posts, with necessary brace timbers, rise from this floor, tl post rows being 8 feet apart. They support a platform which serv as a bridge. Between the upper posts are the gates, twenty in al which slide vertically in grooves formed by spiking additional timbe to the upper faces of the posts. Each gate has a stem to which pow is applied by means of a lever. Every fourth gate is provided wit two vertical revolving shutters, so connected with it that when close they will not interfere with the free movement of the gate. The shutters are intended to be used when the water pressure on the gat is so great as to interfere with their being raised. The cost of tl head gate was $3,000. Lumber at the gate costs $28 per thousand fe board measure. About 60,000 feet of lumber were used in its constru

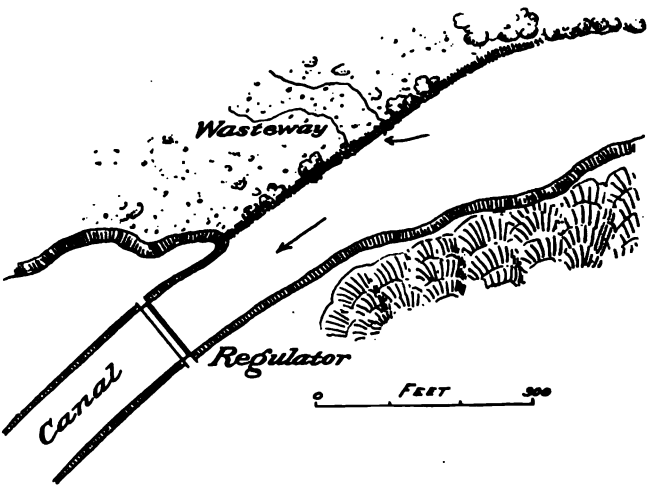


FIG. 15.—Head of 76 Canal.

tion. The check weirs in use on this canal system (there are nine c the Traver branch within 7½ miles above Traver) are simple but ser iceable structures. They are of the flashboard type. Spaces betwe posts 4 feet apart, which rise from a plank floor, are closed by loo boards sliding in grooves. The cost of a check weir across a canal feet wide is reported to have been $275.

Occasionally a firm hardpan layer that would not yield readily blasting or plowing had to be broken up into long, narrow strips wi gads. A spur of rotten granite was crossed at one point, and the ma mum cut through it was 28 feet. Throughout 5 miles below the reg lator large cobbles—some 1 foot in least diameter—had to be remov from the material loosened by the plow before scraper work could co mence. The ground containing these cobbles was first trenched wi large road plows drawn by 12 to 16 mules. The plows were steadi by means of long saplings extending out on either side.

along the base of Campbells Mountain, throughout the 3 or 4 miles to Wahtoke Creek, the surface soil into which the canal was cut is a bog—a dark-colored, light soil, which contracts, cracks, and crumbles in drying. When wet this soil is soft and sticky. This material came to be used in many places for the canal embankment, which often crossed depressions, and when water reached these it rapidly permeated the spongy bank and soon found vents that led to the speedy destruction of the embankment. It cost about $6,000 to get the water in the canal as far as Wahtoke Creek.

The canal was constructed in 1882. This was before the enactment of the irrigation-district law. Its construction was undertaken by a private corporation, on a plan which was very satisfactorily carried out. As soon as the feasibility of making the diversion of water from the river was assured, about 40,000 acres of land on the plains to be commanded by the canal were bought for the corporation at prices less than $10 per acre. These lands were offered for sale as soon as the canal came into service, at prices somewhat in excess of the added cost of canal construction. It was proposed to establish each year a schedule of prices to be adhered to for a year, but demand for land was so great at the prices fixed during the first few years that the lands were

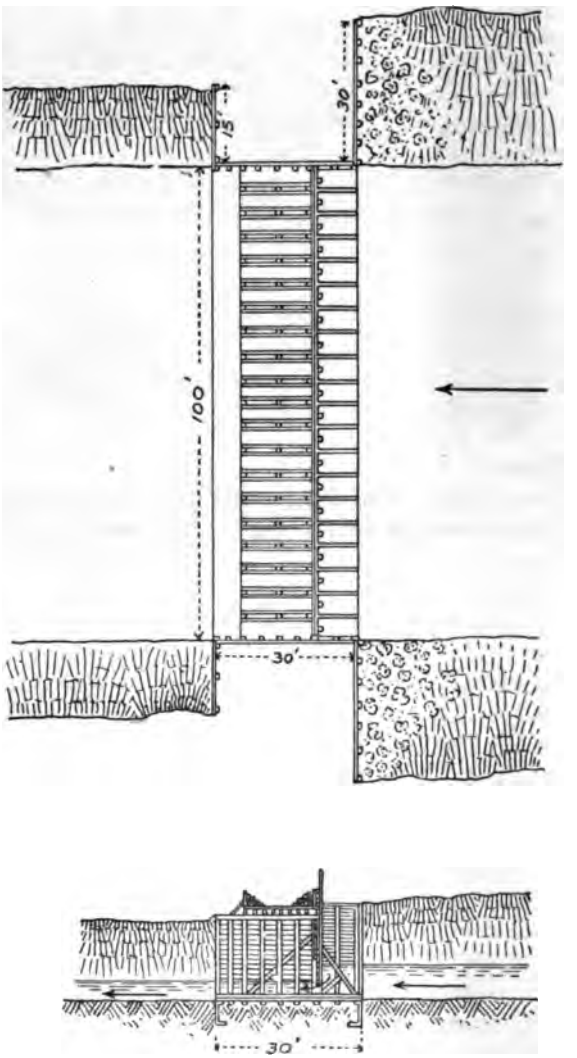


FIG. 16.—Plan and elevation of regulator for 76 Canal.

repeatedly withdrawn from sale. The expenditure of less than $300,000 in four years thus created and increased values to an estimated amount of over $800,000, not including increased values of properties in which the canal company had no direct interest.

Before the canal was sold to the Alta irrigation district, water rights were issued to purchasers of all lands sold by the canal company and were for sale to others owning land in the district commanded. A water right was defined as 40 miner's inches of water, and was located upon some particular 40-acre tract of land, of which it became an appurtenance. Each full water right was made liable to an assessment of $16 per year to cover expenses of canal management. The price of a water right was fixed at $200.

The Alta irrigation district was formed in 1888 and was made to include the lands to be irrigated with the 76 Canal. It extends southerly from Kings River to and even beyond Cottonwood Creek and westerly to within a mile of the eastern border of the Kings River delta, and has an area of 130,000 acres. Two years later the district purchased the canal and its branches, paying therefor $410,000 in bonds of the district. This covered the repayment to holders of water rights of the amounts which they had paid for them.

The northern apex of the Alta irrigation district is at the western base of Tchoenimne Mountain, where a second river bottom, 10 to 20 feet higher than Centerville Bottoms, lies below the 76 Canal. From the second bottom, which is long and narrow, having an area of nearly 7,000 acres, there is an abrupt rise of 20 to 35 feet to the upland or main east-side San Joaquin Valley plain. The surface of this plain drops away gently from the base of the hills southwestward toward the valley trough. Its slope is at first about 10 feet to the mile, but this becomes gradually less and is only 6 feet to the mile near Traver. The only notable break in the surface of this portion of the valley plain is made by Wahtoke Creek, which, in its southwesterly course from the foothills of the Sierra Nevada, passes to the east and south of Campbells Mountain and discharges into Kings River at the Narrows. It flows in a wide, deep gorge from foothill base to the river.

Southward from Wahtoke Creek are several minor creek channels, which carry water only in very wet seasons and sink or spread before reaching Cross Creek.

After the purchase of the 76 Canal system, in 1890, more branch canals were necessary. These were constructed, under the direction and supervision of the district engineer, by private parties, from whom they were then bought, payment being made with irrigation-district bonds. About 150 miles of branch ditches were thus constructed, at a total cost in bonds of $133,000. The entire district is now covered with branch canals, which are rarely more than 2 to 3 miles apart.

The area of land to whose surface water is now actually applied is estimated by the district officers at 40,000 acres. The distribution of

A. FURROW IRRIGATION FOR CITRUS FRUITS.

B IN ALTA IRRIGATION DISTRICT, IN LATERAL ABOUT TWO MILES WEST OF TRAVER.

water is in charge of a canal superintendent and seven assistants, called "ditch tenders."

The soil of this region is for the most part a sandy loam, with coarse, clayey sands near the eastern border of the district, and finer, lighter sands toward the west. Much alkali is in the soil to the west and southwest of Traver. Hardpan subsoils are common at 2 to 5 feet below the surface. Ground water before 1883 was at 20 to 60 feet. It was nearest the surface in the southern and western portions of this district. Irrigation is causing the water table to rise, particularly near Traver.

Water is usually available for irrigation from February until the end of July. Where used on a small scale, land is occasionally prepared for it by subdivision into small rectangular checks. This was the preferred system when water was first introduced into the vicinity of Traver. Now the contour-check method of irrigation is finding more favor. The reason for this preference seems to be due, not only to reduced first cost, but also to the fact that it requires less cutting down of surface soils than the level-check method of irrigation. The uncovered subsoils are often found to be relatively barren. Orchards, vineyards, and summer crops are quite generally being irrigated by the furrow method. Grain land, if wet at all, is irrigated by spreading water over the surface from small ditches or furrows, usually before sowing.

Morrow Ditch.—This was one of several small private ditches south of the main channel of the river which were in use for the irrigation of small tracts of land in Centerville Bottoms. It was constructed about 1865.

Dennis & Jordan ditch.—This is another private ditch, now having the appearance of a natural arm of the river. Its course is southwesterly from Patterson Slough, from which it takes water about 1½ miles below the regulator of the 76 Canal.

Kincaid Ditch.—This name is applied to an eastern arm of Patterson Slough, the head of which is about one-half mile below the head of the Dennis & Jordan Ditch. It supplies water to the Dennis Ditch and to Finks Branch.

Glenn and Barton ditches.—These are two other small private ditches, which seem to have been in use about 1879 on lands of Centerville Bottoms, but which have probably been abandoned.

Emigrant Canal.—This canal may be classed as almost out of service. It was the uppermost of the north-side Kings River canals of the lower group. It took water from the north side of Cole Slough about 2 miles below the head of that arm of Kings River. Its course was southwesterly, nearly parallel with Cole Slough, for about 3 miles, thence northwesterly to and beyond Wildflower. The canal was constructed by an organization of farmers who wanted water for their lands, and was to be managed for mutual benefit. They formed a cor-

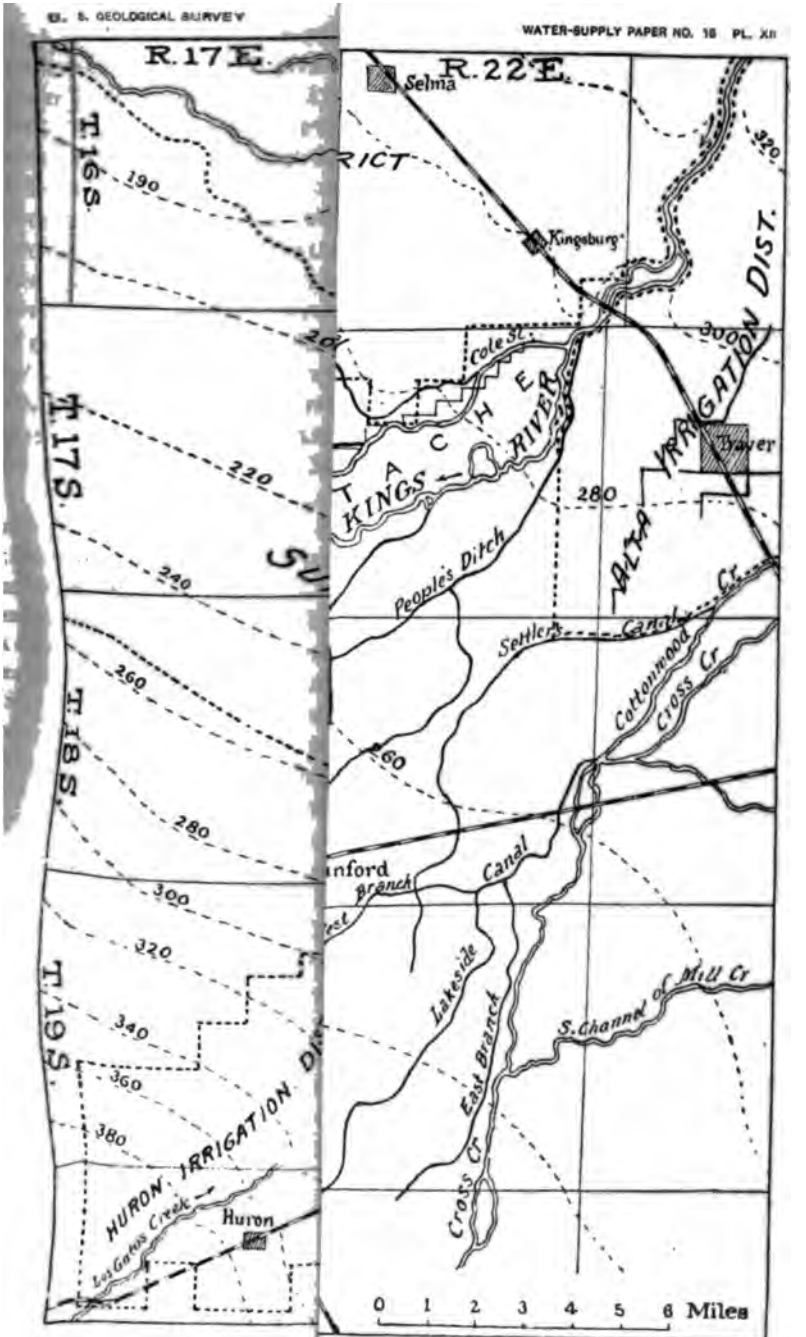
poration late in 1875, with a capital stock of $20,000, in shar $1,000 each. All expenses of construction, maintenance, and man ment were met by assessing stockholders. Each stockholder was tled to the use of canal water in proportion to amount of stock ow He was allowed to sell or rent his water at pleasure. Each consu of water was required to build his own delivery gate, subject to approval of the directors. The distribution of water was in charg a canal superintendent, which office was annually let to the lo bidder. The canal had a bed width of about 16 feet, and delivere water to a number of branches, which covered about 7,000 acres. canal company became involved in litigation and failed to main control of its head works, so that finally it received water only du high stages of the river, and generally for such short periods of that those dependent upon it soon found it to their advantage to se additional water from the Centerville & Kingsburg or the Fowler Sw canals.

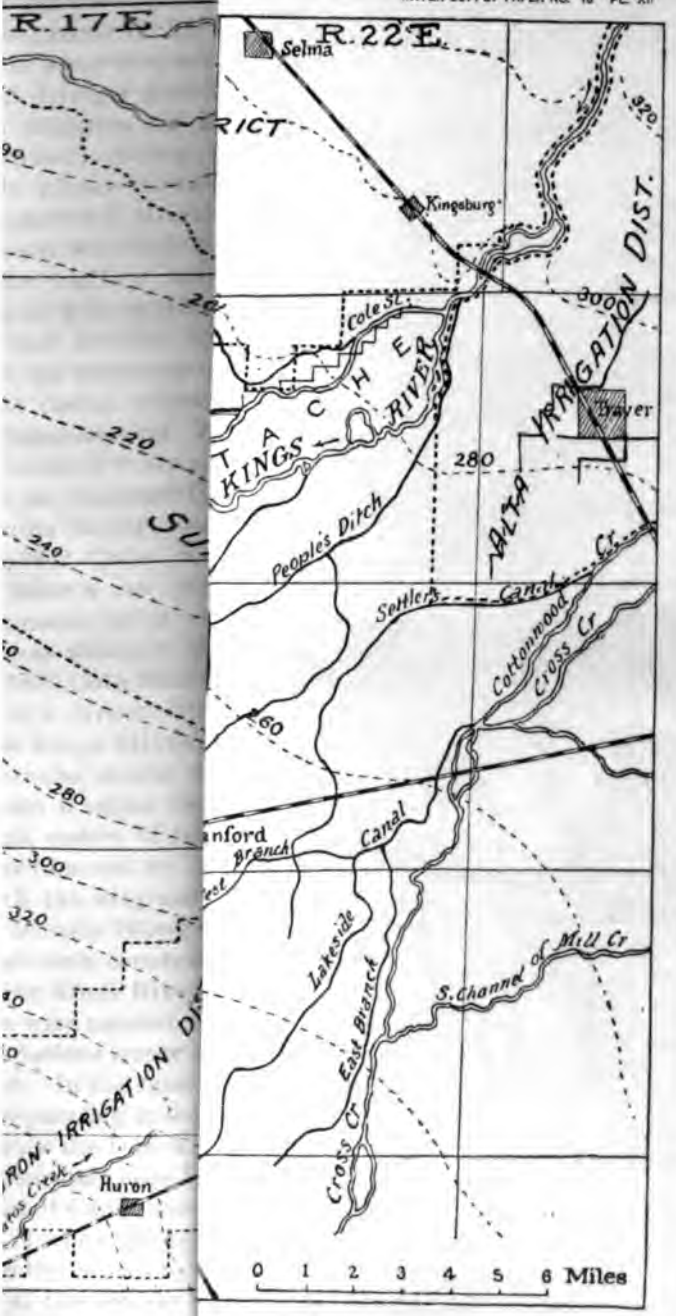
Liberty Canal.—This is a canal which was constructed about from the north side of Murphy Slough, its head being about 5 above Riverdale. After experiencing much trouble in obtaining isfactory supply of water from Murphy Slough, which is the exte of Cole Slough, the original head of the canal was abandoned s years ago, and it was extended easterly about 7 miles to a conn with a high-water escape way from Cole Slough known as the S land Canal. The inflow into the canal is controlled by a reg placed in the Sutherland Canal about one-eighth mile from Slough. The canal is about 15 miles long, has a westerly course miles and thence a northwesterly course, is about 20 feet wide, a branches cover about 5,000 acres to the north of Riverdale.

Millrace Canal.—Where Murphy Slough leaves the grant knc the Rancho Laguna de Tache, about 1½ miles above Riverdal gates or flashboard weirs have been built across the two chan the slough. These force its water through a short canal into a natural channel, known as the Old North Channel, from whicl reclaimed half a mile below and is carried westward and thence ward in the Millrace Canal. This canal is used for the irriga lands near Riverdale and westward thereof. It is essentially a water canal. The canal proper is about 6 miles long. It has width of about 14 feet.

Reeds Ditch.—One of the old ditches on Kings River is the Ditch, whose head is on the north side of Murphy Slough, a miles below Riverdale. It is a small private ditch, having a cc little north of west. Its bed width is about 5 feet; its length a miles. It irrigates a few acres of land near Elkhorn.

Laguna de Tache Canal (Grant Canal).—The "River Ranch," includes the Rancho Laguna de Tache, has an area of about ---es, *embracing* within its boundaries nearly all of the delt


R. 17 E.
R. 22 E.
Selma
Kingsburg
T. 16 S.
T. 17 S.
T. 18 S.
T. 19 S.
190
200
220
240
260
280
300
320
340
360
380
Cole St.
KINGS RIVER
TACHE
ALTA IRRIGATION DIST.
Dover
People's Ditch
Settlers Canal
Cottonwood Cr
Cross Cr
Canal
Lakeside
East Branch
S. Channel of Mill Cr
HURON IRRIGATION DIST.
Huron
Los Gatos Creek
0 1 2 3 4 5 6 Miles


R. 17 E.
R. 22 E.
Selma
Kingsburg
Cole St.
KINGS RIVER
People's Ditch
Settlers Canal
Cottonwood Cr.
Cross Cr.
Lakeside Canal
East Branch
S. Channel of Mill Cr.
Cross Cr.
ALTA IRRIGATION DIST.
Huron
0 1 2 3 4 5 6 Miles

reaching the vicinity of Elkhorn. Its projectors subsequently filed a claim to water and incorporated under the name of the Vanderbilt Canal Company. Settlers in the vicinity of Riverdale and Elkhorn claim that the original cut was entirely artificial, while owners of the Grant Canal claim that the cut is merely the result of cleaning out the head of a natural water course. After the Vanderbilt Cut had become one of the channels for Cole Slough water, the Grant Canal was extended upstream across Murphy Slough to the Vanderbilt Cut, from which it has ever since taken its water. The turning of the flow of Cole Slough into the Vanderbilt Cut is accomplished by means of a dam of earth across the old south channel. From the cut water is turned into the head of the canal by means of a second earth dam, and, flowing in the canal, it crosses Murphy Slough between two dams, of which the lower or westerly one is an overfall dam of brush work, serving as an escape

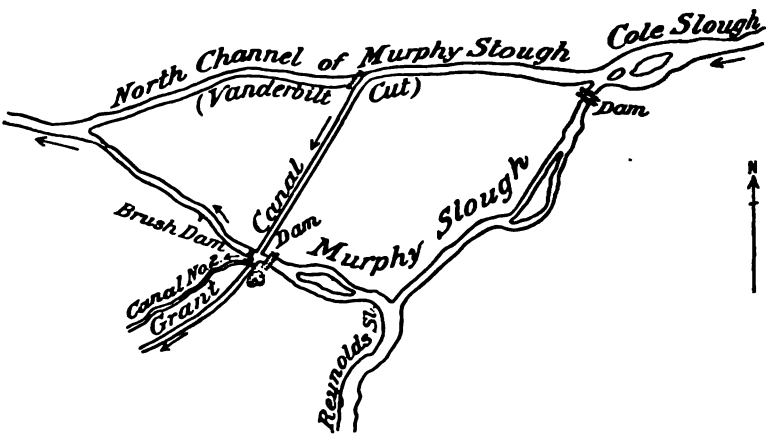


FIG. 17.—Head of Grant Canal.

way for surplus waters and feeding the lower sections of Murphy Slough.

The Laguna de Tache Canal has a westerly course along the northern bank of the main channel of Kings River, from which it is generally less than one-half mile distant. At 5½ miles below its head it crosses a water course known as the James Canal, on an earthen dam, and its direction becomes southerly. It reaches the river bank within a mile and closely follows it southerly for about a mile farther on its course, crossing and closing the head of another delta channel of Kings River known as Old North Fork.

The upper section of the main canal for a distance of about 4 miles from its head has a bed width of about 30 feet and carries water 2 to 2½ feet deep. The distributaries from the main canal are for the most part natural channels, many of which have long ago ceased to serve as delta channels and are mere swales on the tops of low ridges. The James Canal is a channel of more recent formation, which has been

utilized as a distributing ditch. By permission of the owners of the Grant a gate for the control of its flow was put into this channel in 1880. It was washed out by the next freshets and was replaced in 1881, but was again destroyed. Thereupon the owners of the Grant permanently closed this channel with the embankment of earth on which the canal is carried across.

The head of Old North Fork, which is another distributary of canal water, was closed as early as 1865 by Messrs. Sutherland and Mann. One of the most important of the lower-river north-side delta channels is the Zalda Canal, which, throughout its lower sections, is generally known as the North Fork of Kings River. It is claimed that the head of this channel is the enlargement by flood waters of a ditch constructed in 1872 by settlers living to the west of the Grant. This channel was reported closed for a time by a dam at the river about 1885. It was at that time proposed to utilize this channel as one of the distributaries of water from the Grant Canal. The course of North Fork is westward in a very direct line toward Summit Lake, but before the lake is reached its main channel swerves northward into Fresno Swamp. The portion of the Kings River delta between the main river on the south and the Zalda Canal on the north marks the very flat summit between Tulare Lake and Fresno Swamp. In the trough of the valley near and to the south of Summit Lake the general elevation of the ground's surface on this summit, the lowest point on the border of Tulare Lake, is about 211 feet above mean sea level. During the last high stage of the lake, in 1868, the depth of water over the general surface of the country at this point was about 6 feet.

The lands of the Rancho Laguna de Tache have always been so well watered that the irrigation works which have been constructed may be regarded as serving primarily to establish a convenient control of the water rather than as works intended to increase the supply. To prevent excessive natural inundation it has been found necessary to erect embankments along the river, also to construct numerous drain ditches from low tracts into natural channels to facilitate drainage. The main irrigation canal supplies water to a large number of distributaries, frequently natural channels, and these in turn to small irrigating ditches, usually 200 to 450 yards apart. As the entire irrigation system lies within the limits of the rancho there has been less study of methods of controlling and distributing water than would have been the case if a large number of consumers had to be supplied, and water measurement has been entirely out of the question.

The Rancho Laguna de Tache has within the last few years become the property of the same parties who control the Fresno Canal. The future of both properties was kept in mind when the owners of the Grant sold Kings River water to the Sunset irrigation district, to be diverted by way of Cole Slough into the enlarged Grant Canal, thence across the Zalda Canal (North Fork) to a proposed reservoir near Summit Lake.

According to the terms of this agreement the district is given a right of way and a reservoir site, and water is sold to it to the extent of 3,500 second-feet, subject to the prior rights of the owners of the rancho and of the Fresno Canal to the extent of 3,000 second-feet, and subject also to the proviso that, for use on the ranch, 500 second-feet may be taken out of the main canal of the district. The price paid by the irrigation district was $250,000 in district bonds, and the aggregate amount of land in reservoir sites and right of way for the canal was not to exceed 2,200 acres.

Crescent Canal.—This canal was constructed in 1885 and 1886, and came into use in 1887 for the irrigation of lands on the western edge of the extreme southern portion of Fresno Swamp. The head of the canal is on the south side of North Fork, about a mile to the north of Summit Lake. The canal, which is about 8 miles long, has a westerly course for about 1½ miles, thence northwesterly, following a grade line with a fall of 6 inches to the mile. It was originally made 50 feet wide near its head; it was 25 feet wide several miles below, and 50 feet wide in its lower sections. The narrow section was cut to the full width in 1892. It was planned to carry water 3 feet deep. A regulator is maintained in the head of the canal. This is a simple structure. Twelve light gates of 1 inch boards close the 4-foot openings between vertical posts which rise from a plank floor. The total width of the regulator between side walls is 50 feet. Its height is about 6 feet. It is weighted on top with a filling of earth. The canal supplies water to several thousand acres of land, most of which lies within the limit of overflow.

The canal is owned by an incorporated company, whose capital stock was fixed at $75,000, in 150 shares. The cost of the canal works has been about $5,000. The canal was constructed for the supply of water to lands owned by its stockholders and not for the sale of water. Each irrigator is required to own one share of stock for each quarter section of land (160 acres) which he wishes to irrigate. All distributing ditches are constructed by the irrigators, who take water from the main canal at the most convenient points.

Peoples Ditch.—This is a canal of the lower group whose head is on the south side of Kings River, about one-half mile above the head of Cole Slough. Kings River at this point flows in a broad, sandy bed to which there is a steep descent of about 15 feet from the level of the main valley plain on the south. The canal follows the margin of this plain for some distance southwesterly as it recedes from the river, and is gradually brought out upon the surface of the plain about 3 miles below its head. At about 4 miles below its head the distribution of its water to its branches commences. The diversion of water from the river is effected at low stages by means of a dam of brush and sand, which is annually repaired or renewed at considerable expense. Until within the last few years the inflow into the canal was controlled by

A. BRUSH DAM AT TIME OF HIGH WATER PEOPLES DITCH.

B. REGULATOR IN PEOPLES DITCH AS SEEN FROM ABOVE.

two regulators, one of which was within a quarter of a mile of the river bank, the other about 2 miles below. These were 24 feet in width and were of the ordinary culvert type with vertically sliding gates.

There is a new regulating gate now in service within several hundred yards of the head of the canal. This is a massive, well-built structure, 38 feet wide between side walls, which supports an earth fill about 2 feet thick, serving as a roadway over the canal. The space between side walls is subdivided by 10-by-10-inch posts into ten bays or openings, each of which is closed by means of a vertically sliding gate. Power is applied to the gate stems by means of a lever, the end of which engages between the teeth of a rack.

The canal has a fall of only about one-half foot in the first 2 miles of its course. The effective fall is increased somewhat by keeping the brush dam at a good height. The canal as originally constructed was 24 feet wide on the bottom, and was intended to carry 4 feet of water.

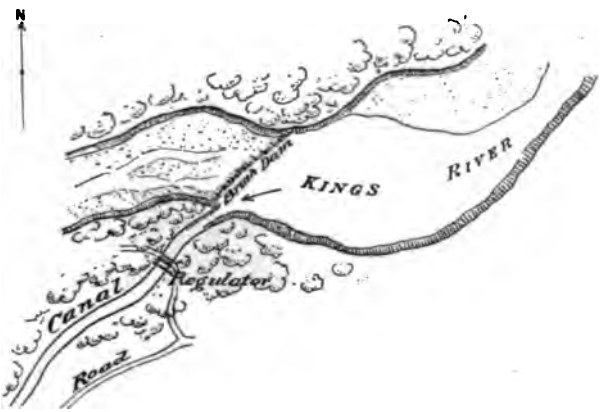


FIG. 18.—Head of Peoples Canal, on Kings River.

Its estimated capacity was about 220 second-feet. Below the second regulating gate the width of the canal increases to 30 feet, and the depth of water to be carried is only 3 feet. These dimensions are maintained for about 4 miles, still in a southwesterly direction and extending well into the northeastern extremity of that portion of the Kings River delta usually referred to as the Mussel Slough country. The canal has three principal branches: West Fork, which terminates near Grangeville; Middle Fork, which passes just to the westward of Hanford; and East Fork, which passes through the southern portion of Hanford and extends to points 4 miles farther south. The aggregate length of the main canal and its principal branches is about 37 miles. This canal was constructed by the Peoples Ditch Company in 1873 and the following years. The first notice of a claim to water was posted late in 1872. The company, formed by settlers who wanted water for their own lands, was finally incorporated in 1873. Its capital stock, originally fixed at $10,000, was soon increased to $35,000, and

later to $100,000. The first cost of the canal was about $50,000 to $60,000. The total cost of canal construction and annual expense account had reached $120,000 in 1881. About $4,000 to $6,000 are annually expended on the dam. The total annual expenditure for management, maintenance, and repairs is about $10,000.

Of the 100 shares into which the company's capital stock is divided about one-third have reverted to the corporation. The remaining shares are in the hands of irrigators, to whom water is delivered in amounts proportional to their holdings of stock. The stockholders are permitted to sell the water to which they are entitled and to ask for its delivery through any of the canal branches. Its use is not restricted to any particular tract of land. The delivery of water to irrigators is usually through gates 1 to 4 feet wide. It is discharged under pressure, the endeavor being to make amounts delivered proportional to stock which it represents, without any attempt at actual measurement. A canal superintendent, with necessary assistants, has charge of the delivery of water. Its equitable distribution depends largely on the judgment of the canal superintendent.

During the years 1883 and 1884 the experiment was tried of selling water to stockholders at $200 per share of stock per year; to those not holding stock an equivalent amount at $300 per year; but it did not prove satisfactory. All stockholders are assessed to meet the annual expenses. The area of land actually irrigated or benefited by the water of this canal is about 25,000 acres.

Mussel Slough Ditch.—This canal, which is now out of service, took its name from one of the delta channels of Kings River which has been utilized for the distribution of water. The canal, unlike the other canals of the Kings River delta, was constructed for speculative purposes. It was proposed to turn a large volume of water from the river into the slough and to derive a revenue from its sale to irrigators. The head of the canal was on the south side of Kings River, about 7 miles below the head of the Peoples Ditch. A narrow cut, the original bed width of which was about 10 feet, was made from that point southerly into and thence southwesterly along the upper portion of the channel of Mussel Slough. The bottom of this cut throughout a little more than a mile from its head was made level, being at an elevation a little below that of the river bed at the head of the canal. The upper portion of this cut, near the river, is over 14 feet in depth. As the canal leaves the river it grows wider, and at about 3 miles from its head the natural channel of Mussel Slough afforded ample capacity for its waters.

Mussel Slough and its principal branches, Sand Slough and Lone Oak Slough, have a general southwesterly course through the delta lands, passing to the west of Hanford and extending to the high water line of Tulare Lake.

To what extent the canal has been in use in the last few years is not

known. It is now out of service, and was not in use for three years preceding 1886. Its head was closed with an embankment of earth in 1884, because at that time the river threatened to destroy the canal head gate and cause inundation of the upper and central portions of the delta.

The canal head gate or regulator was placed in the deepest portion of the cut, near the bank of the river. It was made 24 feet wide, the space between its walls being divided into a number of openings by vertical posts, between which gates slid vertically. The structure had the usual upper platform loaded with earth, making a roadway 20 feet wide. Its floor length was 40 feet. Sheet piling 8 feet deep was used on the upper floor line and for 12 feet under each wing; sheet piling was also driven 6 feet deep across the canal, on the middle line of the floor and at its lower edge.

Throughout the length of the natural channels utilized for the water of the Mussel Slough Ditch, check weirs or gates were constructed with which to hold the water surface at or above the surface of the ground. Irrigation was to be accomplished by subirrigation on a large scale. The charge for water ranged from 31¼ to 62½ cents per acre per year. There was little or no restriction as to the amount of water to be used by individual takers of water, and the principal duty of the canal superintendent seems to have been to prevent those from taking water who had no intention of paying for it.

The total cost of the canal works connected therewith was about $50,000 to $60,000.

This canal was constructed in 1875. Its promoters formed a corporation with a capital stock fixed at $500,000, divided into 10,000 shares, all of which remained in the hands of a few persons. The canal capacity was about 200 to 300 second-feet.

Last Chance Ditch.—One of the most important of the Kings River delta canals is the Last Chance Ditch, whose head is on the south side of the river about 2½ miles by river above Kingston. The course of the canal and its branches is southerly. It commands a district 3 to 4 miles wide, extending from near the river southerly to near the high-water line of Tulare Lake. Grangeville lies at the eastern border of this district and Armona well within it. Lands along its eastern border are commanded jointly by this canal and the Peoples Ditch. A natural cut through the south bank of the river, known as Leinberger Slough, has been utilized for the head of the canal. This slough is really one of the lower delta channels of the river. The canal is cut from its south bank about 150 feet from the river. The canal regulator is about 800 feet below its head. The water entering the head of the slough is divided, a portion flowing down the natural channel, the rest entering the head of the Last Chance Ditch. The canal has a bed width of 25 feet, and was designed for a 4-foot depth of water. Its course is a little west of south for 7 miles to a point half a mile west of

Grangeville, where it is divided into two principal branches. One of these has a southwesterly course and at the end of 3 miles drops its water into a natural channel which has a southerly course and connects with Mussel Slough. The other branch within a mile is again divided, forming a middle and an eastern branch, both of which extend far toward the south. These main branches were all made 16 feet wide on the bottom and were planned to carry water 2½ feet deep. The gradient of the canal is about 1 foot to the mile.

The original canal regulator, which was in service from 1874 to 1880, was made 18 feet wide between side walls and is reported to have cost $5,600. It was replaced by a second one, 20 feet wide, at a cost of $3,000. The removal of sand from the canals, which had been swept in by high waters during the period when there was no gate at the head of the canal, is said to have cost $3,000. The regulator is of the

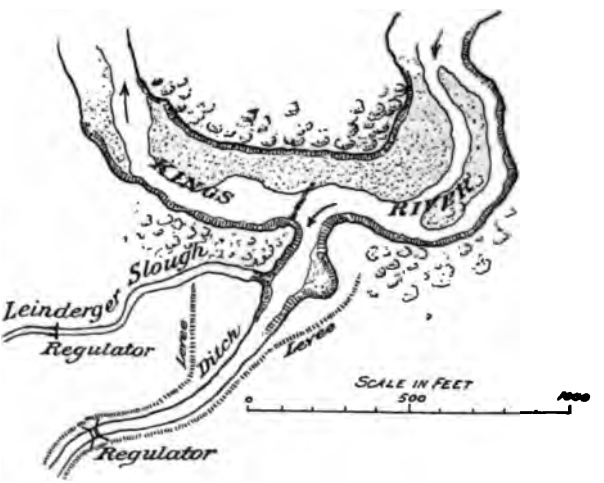


FIG. 19.—Head of Last Chance Ditch, on Kings River.

ordinary box type. From a floor 30 feet wide and 36 feet long (in the direction of the canal) side walls rise to a height of 14 feet. Between them are posts which support a platform or bridge that affords convenient access to the gates. The gateposts divide the space between side walls into six openings, each of which is closed with planking permanently from the top down to about 5 feet from the floor. The remaining open space is controlled by sliding gates having a long stem or standard to which power is applied by means of a lever engaging in the teeth of a rack. The gateposts are braced from below. Long wings extend diagonally into the canal banks above and below the gate. The upper ones are 14 feet high; the lower ones about 4 feet. Tongued and grooved 4-inch sheet piling was driven 10 feet deep along the upper edge of the floor and under the upper wings. Along both sides of the floor, across the canal at its lower end and well out under

A. LAST CHANCE CANAL REGULATOR, AS SEEN FROM UPPER SIDE.

B. LAST CHANCE CANAL REGULATOR, AS SEEN FROM BELOW

the lower wings, 3-inch tongued and grooved sheet piling was driven 8 feet deep. Piles were driven to a depth of 12 feet under the braces to gateposts. Floor sills, piles, and main posts were 8 by 8 inches; side wall posts, 4 by 8 inches. The floor is of 3-inch planking, covered by 1-inch boards. Side walls are of 2-inch material, also covered with 1-inch boards. Back filling was selected sand. The bed of the canal below the gate was protected by a layer of brush, pinned down with stakes 6 to 8 feet long.

The Last Chance Ditch was constructed in 1873 and 1874 by a company known as the Last Chance Water Ditch Company, which was organized and incorporated by the farmers to whose lands it was to supply water. The original 30 shares into which a capital stock of $30,000 was divided were a few years later increased to 60 shares, and the capital stock was also doubled. Only about two-thirds of these shares are in the hands of stockholders. Until 1886 all operating and repair expenses, about $4,000 per year, were assessed upon the stockholders, and water was delivered to these without charge in proportion to amount of stock owned or controlled. Since that time water has been charged for at rates fixed annually, generally $1.50 per acre, in order to make actual users of water bear an increased portion of the canal expenses.

There has been no system of water measurement introduced. Water is delivered to irrigators through a great variety of gates; generally, however, through openings under pressure. A canal superintendent and an assistant have charge of its distribution. During the high-water period an attendant is kept constantly at the head gate. The canal ordinarily flows from the beginning of February to the end of July. About 20,000 acres of land are dependent upon it for water.

Lower Kings River Canal.—This is another south-side canal in the Kings River delta. Its head is about 6 miles below Kingston. Its course is southerly, and the district it commands lies just westward of that of Last Chance Ditch and has Lemoore in its center. The canal is cut southerly from the river, and at the end of its first mile reaches Leinberger Slough, into which its water is dropped. It is reclaimed from this slough less than half a mile below that point, and thence has a southerly course to within about 3 miles of Lemoore. Two principal branches, each about 16 feet wide on the bottom, extend toward the south from this point. Near the river the main canal crosses high-water sloughs fed by the over-bank flow from Kings River. These natural channels have been closed by dams on the lower or west side of the canal. Their water, together with other water escaping from Kings River above the head of this canal, is received by the canal and carried to Leinberger Slough. At the point where the canal leaves this slough a waste way 260 feet long is maintained, through which surplus waters are discharged. This is a very light timber structure, which is reported to have rendered good service. Some years ago it was proposed to

supplement it with a second waste gate, 200 feet long, nearer the head of the canal. The main canal regulator or head gate has been set close to the river bank. A secondary regulating gate is just below the Leinberger Slough waste gate. Each gate is about 40 feet wide between side walls. The head gate is arranged similarly to that described for the Last Chance Ditch. The main canal is about 40 feet wide. It was built on a very light grade. The bottom of the canal, commencing at the same elevation as the river bottom, was reported level throughout the first 5 miles of its course. The effective gradient slope of water surface when full is reported at 4 inches to the mile, with a 2½-foot depth at the forks of the canal. Work on this canal commenced in 1870. Twenty-seven persons joined in an agreement under which the canal was to be constructed, and which provided that each person was to bear a proportional part of "all expenses above" his "further boundary." It was at first made 18 feet wide on the bottom near its head, and 10 feet wide at 5 miles below its head. It was enlarged in 1873. The first head gate was constructed in 1872 and is reported to have had a width of 18 feet. It was washed out the following winter and was at once replaced by another, which was in use till 1876. After this second regulating gate was swept away the canal was operated without an upper gate until 1879, when the third regulator, 40 feet wide, was built, at a cost of about $3,000.

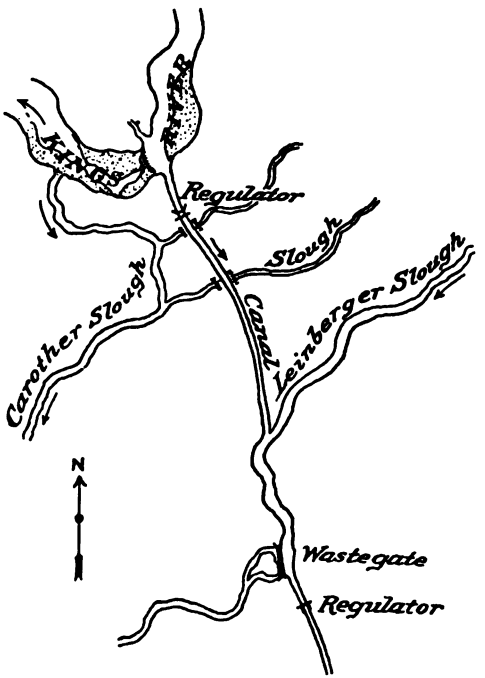


FIG. 20.—Upper section of Lower Kings River Canal.

The canal was built and is owned by those to whom it supplies water. Its owners, who had organized as a company, formed a corporation in 1873, with a capital stock fixed at $30,000, in 100 shares.

The water in the canal is supposed to be represented by those shares of stock which have been issued, which is a little more than one-half of the full number of shares of capital stock. It is sold by directors of the company, but only to the extent of the shares which have been issued. Each of these represents a proportional part of the flow of the canal until such part exceeds 144 miner's inches. Water in excess of

amount is at the further disposal of the company. Stockholders referred purchasers of water to the extent of the shares owned by and provided the application for water is made preceding a fixed Each share of stock is supposed to represent sufficient water for cres. The annual cost of canal management and repairs is about 0 to $2,000. The first cost of the canal was about $30,000. For ber of years after organization all expenses were assessed upon tockholders, but, this proving unsatisfactory, the plan of selling was introduced, with good result. The extent of irrigation from anal may be noted at about 10,000 acres.

ads Canal.—Lands to the west of Lemoore to the extent of 3,000 00 acres are dependent in large measure upon the flow of the ds Canal for irrigation water. This canal in its upper section has ppearance of a natural high-water channel. From a point on the side of Kings River about 1½ miles below the head of the Lower s River Canal, this channel has a southwesterly course for 2 miles rother Slough. It is known as Wrights Cut, and is claimed to be nlargement of a small ditch constructed many years ago. The ds Canal is carried southward from the termination of this cut at ther Slough about 7 miles. It has a bottom width of about 16 nd carries water about 2 feet deep. Water is taken from Carother gh so long as the slough has a sufficient flow; thereafter a little is done at the head of Wrights Cut to turn more water from the into the canal.

e canal is reported to have been in use since 1868. It replaced a ditch, about 5 feet wide, which had been in use some years before. anal belongs to a number of farmers, who by mutual agreement rtion its water and expense of maintenance among themselves. e is no regulator at the head of the canal. Water is taken by irrigator as he may require it. No water is sold, and all contribu- toward expenses of maintenance are voluntary, as there are no s of enforcing their payment.

ere are times when much more water flows in Wrights Cut than e carried by the lower sections of the canal. The surplus reaches n Slough through Carother Slough, thence Esrey Slough and natural channels, and is finally reclaimed for the irrigation of about 4 miles southwest of Lemoore.

obs Canal.—This is a private high-water canal, constructed about ears ago for the benefit of lands along the northern margin of e Lake, below the lake's high-water line. Its head is about 6 southwest of Lemoore.

SUNSET IRRIGATION DISTRICT.

the construction of the works for this district has not commenced, ef reference to the district and its proposed source of supply will e. The district covers the lowest portion of the west side plain n Joaquin Valley, extending northward from the southern limit

of Tulare Lake about 70 miles, to within 3 miles of Las Juntas, th point where Fresno Slough unites with San Joaquin River. Its widt ranges from 1 to 12 miles and its area is about 363,500 acres. The di trict organization was completed in March, 1891. After estimates o the cost of works necessary for its irrigation were made, a bond issu of $2,000,000 was voted. Of this bond issue $250,000 were paid fo water rights, reservoir site, and rights of way.

The plan of irrigation involves the construction of a large reservoi in the vicinity of Summit Lake, which is to be formed by the constru tion of two embankments of earth, each crossing the valley troug from east to west. Water is to be supplied to this reservoir fro Kings River by way of Cole Slough, the enlarged Grant Canal (whic is to be used in common with the owners of the Grant), and a can extension to be carried from near the lower end of the Grant Canal i a direction a little south of west across the North Fork of Kings Rive to the reservoir site. From the reservoir there is to be a canal nort ward and a second one southward, both supplied with water by gravit flow from the reservoir. In addition to these two low-level canal there are to be eight other canals, four northward and four southward each 11 feet higher than the one next below, and each to be supplie with water by means of centrifugal pumps arranged in four pumpin stations.

The specifications drawn for this work do not indicate how the sev enteen pumps that are prescribed are to be apportioned to the severa pumping stations. The pumps are each to have a capacity of 10 second-feet raised 11 feet. The district canals are to be of sufficien size and length to carry water to all parts of the district. The require ments concerning canal dimensions are not made specific; they are t be suitable to the service required, which, it appears, remains to b determined as work progresses. The aggregate length of the propose district canals is estimated at 500 miles. The contract for the constru tion of the canal system complete, including the necessary pump engines, and boilers, has been let for $1,500,000, and the same perso who has taken this contract is also the purchaser of the bonds of th district. It is distinctly expressed in the contract that in case th bonds are not taken the contract may, at the option of the distric directors, be set aside. The contract was made several years ago, an was conditioned upon a decision by the Supreme Court of the Unite States sustaining the irrigation-district law. This having been ren dered, it is understood that the contractor has now declared his readi ness to proceed with actual construction work.

THE FRESNO PLAINS.

DESCRIPTION.

The main canals which divert water from the north side of Kings River at or near the upper end of Centerville Bottoms and command that portion of the San Joaquin Valley plain in the vicinity of Fresno, Selma, and Kingsburg may be designated the Fresno group of canals. These canals are the Kings River & Fresno Canal, the Fresno Canal, the Fowler Switch Canal, and the Centerville & Kingsburg Canal, all of which have already been described. Their branches extend throughout an area of nearly 400 square miles, which before canal construction was a treeless, sandy plain. It was almost devoid of natural water courses. The few creeks whose periodical flow reached the eastern border of the plain to the north of Kings River were soon lost, and there was no sign of a channel beyond their "sinks." The natural water courses in the eastern portion of the district under consideration, in their order northward from Kings River, were Burns Slough, Kip (Lone Tree) Slough, Mud Slough, Mud Creek, Fancher (Fanshaw) Creek, Red Bank Creek, Frolic Creek, and Big Creek. Of these, the first three lie to the south of the point of the mountain which with gentle slope extends westward on the north side of Kings River to a point 2 miles north of Centerville. Burns Slough is a tributary of Kip Slough. The sink of Fancher Creek, now a fine vineyard, is about 4 miles east of Fresno. The waters of Red Bank and Frolic creeks, after uniting in one channel, spread and disappear 6 miles east of Fresno, near where Temperance Colony has been established. The sink of Big Creek is several miles to the northwest of Fresno.

Some of the channels of these creeks have been made part of the canal system. Portions of Burns Slough and Big Creek are utilized for water of the Kings River & Fresno Canal. Other portions of Burns Slough, Kip Slough, Mud Slough, Mud Creek, and Fancher Creek are now the main channel of the Fresno Canal. Lone Tree Creek is an important distributary of the water of the Fresno Canal. By the utilization of these natural water courses the canals are made the recipients of the drainage water from the foothills, and are, at time of considerable rainfall, taxed beyond their capacity, and have repeatedly in the last few years inundated parts of Fresno and vicinity. A proper drainage system in this locality is now recognized as a necessity, and no doubt will be planned and executed in the near future. The general slope of the Fresno irrigated region is from northeast to southwest, at the rate of 5 to about 7 feet to the mile. The surface of the country for the most part is very smooth, though broad belts of hog-wallow or knolly ground are occasionally found. The knolls of these hog-wallow belts are generally 1 to 2 feet in height, rarely as high as 3 or 4 feet. The knolls are close together, and diameters at the

caused a rise of the ground-water plane, and that it will continue to influence its elevation, the question whether water has been or is being lost by the apparent duplication of canals remains an open one. Certain it is that were all water drawn from the river in one main canal it would still be necessary for branch canals to extend to every portion of the irrigated region. The ground water would be expected to rise equally as rapidly as at present, and the only marked saving of water would result from the exposure of a decreased surface area of flowing water to evaporation, and, in this special case, from a possible reduction of the flow of water from the main canal through subsoils back into Centerville Bottoms.

LOSS OF WATER BY EVAPORATION AND SEEPAGE.

The loss by evaporation, when expressed in second-feet, is small for any of the canals of the Fresno system when compared with their capacity or with the total loss by infiltration into subsoils. The State engineering department made experiments to determine the rate of evaporation in this vicinity. The observations were extended through a four-year period at Kingsburg.[1] The greatest total monthly evaporation during the four years 1881 to 1885 occurred in August, 1883. The amount of water evaporated in that month from an open sheet of water was equivalent to a depth of 0.945 foot. At this extreme rate per month the loss by evaporation from the surface of the Fresno Canal throughout the 24.5 miles of its length, if the canal be considered 60 feet in average width (which allows ample margin for wet banks), would have been equivalent to a continuous flow of 2.8 second-feet.

To determine the amount of water which sinks into the subsoils from the canals of this region, measurements of their flow were made in the summer of 1882 by the writer, then assistant State engineer. The results will be found on pages 76 and 77, in tabular form. For the better understanding of the results presented in the tables it is necessary to refer briefly to the character of the several canals and ditches whose flow was at that time measured.

The Kings River & Fresno Canal at its head cuts at once into the upland to the north of Centerville Bottoms. For a long distance it is nearly parallel and sometimes very close to the Fresno Canal, with water surface at a higher elevation than that of the latter. At many points the canal carries its water above the surface of the adjacent plain. Ponds and swamps were kept full of water which the canal lost by percolation. The canal, throughout the last few miles of its first 12, lies in a good channel with hardpan bed. The loss of water in the upper 1[illegible] miles was at the rate of 3.77 second-feet per mile. In 1882 the section of the canal below Red Bank Creek was poorly maintained. Water was allowed to spread out in ponds, and a former channel lying parallel to the present canal, and generally very close to it, frequently carried part of the canal waters. No lands on this section of the canal were

[1] See Physical Data and Statistics, Wm. Ham. Hall, State Engineer of California, p. 279.

grapes, fruits of all kinds, including citrus fruits, corn, melons, etc. Two of the canals irrigating lands near Fresno have a perennial flow; the others divert water only during medium and high stages of the river, generally from January to August, inclusive. The aggregate capacity of the four canals already enumerated is about 1,550 second-feet. None of them, however, flow for long periods at their maximum capacity. The demand for water is, fortunately, greatest at the time when the flow of the river is greatest, during spring and the early part of summer.

THE CANAL SYSTEM.

All of the canals of the Fresno system take water from Kings River above Centerville. Each of them is out upon the upland before passing that village. Such duplication of canal work seems unnecessary.

Each of these canals was constructed for the purpose of supplying water to some particular point or place of use; thus, the Fresno Canal to irrigate lands in the immediate vicinity of Fresno, the Kings River & Fresno Canal to irrigate a strip of land just north of the Fresno Canal, the Centerville & Kingsburg Canal to irrigate lands near Selma, and the Fowler Switch Canal to supply water to the vicinity of Fowler and lands westward from that point. By reason of such special requirement in each case, each canal received an alignment as direct as possible from the proposed source of water to the point of delivery. The main canals were therefore constructed without attempt to hold them to any uniform grade. They have alignments in the direction of the greatest fall of the country, and generally, when their grade is excessive, check weirs are used to prevent erosion. The flow of water in the Kings River & Fresno Canal and in the Centerville & Kingsburg Canal is thus regulated. The regulation in the case of the Fresno and the Fowler Switch canals is only partial. The water of the former of these canals in Fancher Creek is generally 8 to 15 feet below the surface of the ground, its elevation being uncontrolled by structures.

The Fowler Switch Canal was constructed on a very irregular grade line. Its grade, when possible, was placed at 1.92 feet to the mile, which was established as the minimum gradient. At some points, however, the grade was no less than 12 feet to the mile for short distances. The dimensions of the canal were calculated on a basis of a fall of 1.92 feet to the mile, and were not modified for heavier grades. Consequently, canal dimensions where this grade was exceeded are excessive. The canal was originally constructed without structures, either for the regulation of its flow or to facilitate diversion. Rapid erosion at once commenced where the canal bed was not in very firm material, and after a short experience a limited number of check weirs were put into it at the points where most required. The structures in this and in the other canals of the district are very light timber affairs, but generally well adapted for the purpose which they are to serve.

In view of the fact that the irrigation of this region has in the past

edge of the bluff from upland to the river bottoms, and less than half a mile distant from it. For several miles the canal is within several hundred yards of the edge of the descent to the bottoms, which lie 20 to 30 feet below the surface of the upland. This location in porous soils and subsoils causes great loss of water by percolation. Nearly 86 second-feet were lost by the canal in less than 6 miles, and in its seventh mile the canal lost 52 second-feet. This last result is based on measurements of the flow of water over two weirs 1 mile apart, apparently in good condition. The average loss per mile from the canal in the 5½ miles of the upper section examined was 15.63 second-feet.

Loss of water from the Kings River & Fresno Canal.

Name of canal or ditch, and locality.	Distance below head of canal, in miles.	Discharge, in second-feet.		Loss between stations, in second-feet.	
		Main canal.	Diverted.	Total.	Per mile.
K. R. & F. C., one-half mile below head	0.50	133.83	.....		
Burns Ditch	1.00	.....	0.25		
Hansen Ditch	4.00	.....	3.23		
Fancher Creek (wastage)	8.00	.....	.30		
Ditch on south side	8.00	.....	4.05	43.36	3.7
Do	9.00	.....	.05		
K. R. & F. C. at Hawkins Weir	9.00	[1]78.18	.....		
Wastage	.....	.....	.15		
Ditch on south side	11.50	.....	2.04		
K. R. & F. C. in Red Bank Creek Flume	12.00	80.40	.....		
Dog Creek (wastage)	13.25	.....	.15		
Ditch on north side	16.00	.....	3.36	16.50	3.48
Eggers Ditch	16.75	.....	4.67		
K. R. & F. C. at Eggers Weir	16.75	55.72	.....		
K. R. & F. C., west line of section 17	17.75	[1]49.56	.....		
Ditch on south side	17.80	.....	3.08		
Scandinavian Colony Ditch	18.25	.....	3.86		
K. R. & F. C. at upper Scand. Col. Weir	18.25	[1]36.74	.....		
Scandinavian Colony Ditch	18.35	.....	1.46		
Do	18.50	.....	2.42		
Do	18.60	.....	1.00	4.44	1.2
Do	18.70	.....	2.39		
Do	18.80	.....	14.50		
K. R. & F. C., lower Scand. Col. Weir	19.00	[1]25.51	.....		
Big Creek (wastage)	19.25	.....	1.50		
Wastage	.....	.....	.20		
Ditch on north side	19.75	.....	.43		
Cooper & Helm Ditch	20.25	.....	8.71		
K. R. & F. C. at Cooper Weir	20.30	11.73	.....		

[1] Approximate.

Loss of water from the Fresno Canal.

Name of canal or ditch, and locality.	Distance below head of canal, in miles.	Discharge, in second-feet.		Loss between stations, in second-feet.	
		Main canal.	Diverted.	Total.	Per mile.
sno Canal at Road Bridge	1.25	381.00	.....		
Centerville Branch	1.75	.....	17.80		
Lone Tree Branch	3.25	.....	69.39	95.51	8.49
Ditch on south side	5.00	.....	.50		
Limbaugh Dam Ditch	12.50	.....	58.72		
sno Canal at Limbaugh Dam	12.50	139.08	.....		
Briggs Canal	14.00	.....	.30		
Eisen Canal	14.00	.....	14.09		
sno Canal at Hobler Dam	14.00	107.88	.....	5.20	.74
Easterby Ditch	17.25	.....	5.60		
Malters Ditch (north)	19.00	.....	6.30		
Malters Ditch (central)	19.00	.....	1.69		
Washington Colony Branch	19.50	.....	45.50		
esno Canal at head of Wash. Col. Branch	19.50	60.40	.....		
Fresno Colony Canal	23.50	.....	14.15	3.81	.95
North Central Colony Canal	23.50	.....	13.88		
Main Central Colony Canal	23.50	28.16	.....		

Loss of water from the Centerville & Kingsburg Canal.

& K. Canal, one-half mile below head	0.50	346.00	.....		
Ditch on west side	5.50	.....	5.06	85.94	15.63
Do	5.75	.....	4.00		
C. & K. Canal at weir, in sec. 23	6.00	251.00	.....		
Garfield Canal	7.00	.....	26.65	52.35	52.35
C. & K. Canal at weir, in sec. 26	7.00	172.00	.....		

Other examples of the loss of water from smaller ditches by percola-
on into the subsoil are the following:

On June 21, 1882, water was being used for the irrigation of a vine-
rd in Scandinavian Colony. The water was diverted from the main
pply canal into a small ditch about 2 feet wide. It was conducted
st alfalfa fields and vineyard lands a distance of one-half mile to the
int where it was being used for irrigation. The soil crossed by the
ch is a sandy loam, with ground water at 8 feet below the surface.
ater was measured at the point of diversion and at the point of
livery in exactly the same way, over small overfall weirs. The
ount diverted was 1.70 second-feet, and the loss in transit 0.79
nd-foot.

On June 14, 1882, the orchard of the Gould Ranch north of Fresno was being irrigated. The water used for this purpose was flowing in a ditch having an average width of 3 feet. One mile above the point where the water was being used the discharge of the ditch was 2.41 second-feet. The loss by percolation before reaching the point of delivery was 1.42 second-feet. Shade trees were growing along portions of the ditch, and its banks were overgrown with grass. The ditch had been in use several years. The soil through which it flows is a sandy loam 5 to 8 feet deep.

On June 26, 1882, a ditch which leaves the Fresno Canal at a point about 4 miles east of Fresno was carrying 5.60 second-feet near its head. One mile below it was receiving 0.03 second-foot, and at other points 0.64 second-foot; its flow 4 miles below its head was only 1.39 second-feet; it had lost 4.38 second-feet. This ditch has an average width of about 8 feet. It carried its water at or a trifle above the level of the ground's surface.

EFFECT OF IRRIGATION ON THE WATER TABLE.

In no other irrigated region in California is the effect of irrigation on the elevation of the water table so plainly apparent as in the vicinity of Fresno. Here, notwithstanding the 6 feet per mile slope of the surface of the country, the subsoils have been and are being gradually saturated with water. When irrigation commenced the loss of water from canals in transit to lands to be irrigated was very great, and after reaching its destination it was found that frequently enough water was put upon the surface of individual small tracts of land to have covered them to an average depth of 15 to 20 feet in a season. Most of this water found its way into subsoils, together with the direct loss of water from the canals. When the first wells were dug at Centerville water was found at about 20 feet. It is now at 10 to 15 feet below the surface, and will probably not rise much higher on account of ample drainage of subsoils and a comparatively free subsurface flow toward the low river bottoms.

Along Fancher Creek, where the creek is used as a canal, ground water was formerly at 50 to 75 feet below the surface; it is now at 12 to 20 feet along the upper portion of the creek, and still nearer the surface near the former sink of the creek, about 4 miles east of Fresno. In the Eisen vineyard, 5 miles east of Fresno, ground water is at 4 to 6 feet below the surface, and drain ditches have been constructed to prevent a further rise. Ground water is found at 4 feet below the surface in many parts of Temperance Colony, Nevada Colony, and Fresno and Central colonies. Throughout a considerable area near Fresno its surface is nowhere more than 16 feet below ground, though before irrigation commenced depth to water was 60 feet. Cellars which were in use until 1884 in Fresno have been condemned because ground water made its appearance in them. Near Selma and Kingsburg

ground water was formerly found at about 30 feet. It is now at about 15 feet. The so-called "Sand Hollow," which is a depression 100 yards to one-fourth of a mile wide, 12 miles long, and about 16 feet deep, having a southerly course and passing about midway between Selma and Kingsburg, was formerly as dry as the surrounding plain. About 1880 the soil in its bed was found to require less moisture to produce crops than the adjacent plain; in 1885 water appeared at the lowest points of the bed, and in June of that year it had risen to the heads of ripening grain. Spots of black alkali now mar its bed and sides, which before were a light sand, apparently free from any excess of alkaline salts.

Near Sanders and at points eastward from there toward Kings River ground water was at 50 to 60 feet before irrigation commenced; it is now at 20 to 30 feet. Here, as in the case of Centerville, proximity to the river, which lies 60 feet below the surface of the plain, may interfere with a much greater rise of the ground-water plane.

In connection with this change in the elevation of the ground-water plane, it must be remembered that the effect of irrigation extends far beyond the points now under ditch. Owing to the slope of its surface and consequent hydrostatic pressure, ground water has a slow but certain motion westward through pervious subsoils of the Fresno region, and may therefore ultimately contribute more or less toward the flow of Fresno Slough and San Joaquin River.

METHODS OF IRRIGATING AND DUTY OF WATER.

At the time irrigation commenced near Fresno the soils and subsoils were dry; depth to ground water was 30 to 60 feet. The ground was porous, generally sandy, sometimes very hard on the surface. When wet it was converted into a bog. Water passed through it readily into the lower strata. Thorough wetting compacted the soil, and it was less liable to conversion into slush after the first and second wettings than before. It settled, as a result of the first wetting, about 8 inches to 1 foot, on an average. Sometimes the full supply of a ditch would flow for hours into a hole formed by the breaking in of the surface. Cavities 20 feet in diameter and 6 to 10 feet deep were occasionally formed in this way, and had to be refilled after the first wetting. The stability of buildings was in danger when water was first brought near them. It is not surprising, under these circumstances, that when irrigation commenced in this region sometimes enough water was run on small areas at one irrigation to have covered them 5 feet in depth, and that enough water was delivered to some of the 20-acre colony lots south of Fresno to have covered them to an average depth of 20 feet. After irrigation was practiced for a time the soils became more compact, absorbed water less rapidly, and a smaller amount of water passed into subsoils. At present the ground water near Fresno is so near the surface that the roots of alfalfa, vines, and trees readily penetrate to

it and soils are kept moist by the water rising from below. Surface wetting has become unnecessary in many sections.

Where land is held in tracts of 20 to 40 acres, as in the colonies near Fresno, the subdivisions of alfalfa, vineyard, orchard, etc., are small. The individual irrigator has but a small area of soil to moisten and, because many irrigators in his neighborhood necessarily require water at the same time, he can probably command but a small flow with which to accomplish his irrigation; consequently he devotes more than ordinary care to the preparation of his land, and if the duty of water is not so high on his lands as it ought to be the good will of the irrigator is not at fault.

The subdivision of land holdings into small tracts and the greedy absorption of water at the outset by the dry soil of the Fresno plains were the prime factors which have controlled methods of irrigation. That which most frequently found favor was flooding in small checks whose surface had been made perfectly level, it being extremely desirable to quickly cover the whole surface to be wet with a thin sheet of water. Drainage appeared unnecessary, as all surplus water delivered into a check rapidly sank into the ground.

The area of these level checks generally ranges from one-quarter of an acre to 2 acres. They are nearly always made rectangular. In alfalfa fields the levees inclosing them are often made so flat and low that farming implements can pass over them. Their position is adapted to the configuration of the country, in order to make the work of leveling as small as possible. Irrigation ditches are so arranged that there is an inlet from a ditch into each check. No drains are provided. All surplus water entering a check is allowed to sink into the soil. The cost of preparing land in this way has been variously stated by the irrigators at from $15 to $50 per acre.

Water is admitted into the several checks through small gates, or occasionally through small sluice boxes, generally constructed of 1-inch redwood. Water in the irrigating ditches is controlled by means of small gates closed by loose boards moving easily in vertical grooves. Sometimes a notched board is set across the ditch, so arranged that the notch can be closed by means of a board upon either side of the notch.

The system of flooding in checks formed by contour levees, which has found much favor with Kern River irrigators, is in use only for the irrigation of large tracts of alfalfa, but where used in the vicinity of Fresno the area of the checks is kept small, rarely exceeding in any case 3 or 4 acres, whereas in the Kern River country the area of a single check frequently exceeds 10 acres. Contour levees are lower for checks of smaller area. The expense of ground preparation is greater, but irrigation is more economical, as with small checks much time and water are saved. The contour levees usually follow closely 4-to-6-inch contour lines. Soils are so porous that irrigation in each *check is complete when* water has been made to cover its whole area,

otherwise there is no deviation from the ordinary practice of irrigation in contour checks.

A third system of flooding is sometimes practiced, though it has not found general favor. It is that of flooding the surface with water from small ditches which are carried along the tops of ridges, or very nearly on contour lines. The water of these is made to overflow one or both banks, and as the thin sheet of water flows over the surface it is guided and to some extent controlled by attendants. Unless the ground is very knolly this system requires but little preparation, except the construction of the irrigating ditches, but the application of water is attended with much greater expense than under the preceding systems. It has been in use for trees and vines as well as for alfalfa.

The furrow method of irrigation is growing in favor. It is practiced in many ways, according to conditions of surface slope and crops to be irrigated. When water is to be supplied to the surface of grain land in large areas it is generally done by the furrow method. Distributing ditches supply water to plow furrows or to small temporary ditches which are placed so close together that water can readily permeate the surface soil, particularly that portion of it which has been loosened by the plow. The furrows may be run on or near contour lines; they may be run in the direction of the greatest slope of the ground's surface, or they may be run in parallel lines between the rows of trees, vines, corn, vegetables, etc. Sometimes this system of irrigation is practiced in level checks for trees and vines in preference to the plan of flooding the whole surface.

When grain land is to be irrigated by the furrow method the furrows are drawn 8 to 12 feet apart in the direction of the greatest slope of the ground, unless this be so great that water flowing in the furrows would cause erosion. Should this be the case they are drawn obliquely to the greatest slope. The irrigating ditches are located on the ridges, if there be any, or on grade at intervals of one-half to one-quarter mile from near the upper edge of the tract to be irrigated, and about parallel with it, to near the lower edge of the tract. The irrigation commences at the highest point of the field from the upper ditch. Water is admitted into a number of furrows, in which it flows until moisture reaches the next ditch; it is then turned into other furrows, until all the furrows between the first and second ditches have been wet. Hereupon the water is turned into the second ditch, another set of furrows is wet, and so on until irrigation is complete.

This system of wetting from furrows is practiced for grain land as well before seeding as when a growing crop is to be irrigated. In the latter case the refilling of small ditches and furrows that would otherwise interfere with harvesting operations adds somewhat to the expense of irrigating.

Under the furrow system of irrigating each attendant ordinarily controls about 1½ to 2 second-feet, and water is generally turned off at

night, owing to the difficulty of controlling it in the dark. On the sandy soil of the region covered by the Centerville & Kingsburg Canal, where this methed of applying water has found more favor than elsewhere, irrigation progresses at the rate of about 2 acres per day for each second-foot of water. Each irrigation is there equivalent to an average depth of one-half foot of water over all the land irrigated. It should be stated, however, that very little wheat land is regularly irrigated on the Fresno plains at points where water must be applied to the surface soil. Irrigation is rather a last resort to save a growing crop which the farmer hoped would mature without irrigation. For the regular irrigation of grain land it is customary to rely upon one irrigation in the fall of the year before plowing. The moisture then absorbed is sufficient to supplement the scant rainfall and make a good harvest reasonably certain.

The lack of natural deep-cut drainage lines through the lands of the Fresno district has already been alluded to. Their absence has made possible the remarkable filling of the subsoils with water and the rising of the ground water to within a few feet of the surface. Whenever ground water is brought so near the surface that capillary attraction and the hygroscopic properties of soil cause water to rise sufficiently high to moisten the surface layers of soil, other methods of supplying moisture become unnecessary. Crops are irrigated if ground water is permanently held at this height, or if it is so frequently raised to this height that the soils under cultivation receive all the moisture required from below. Where water is thus brought within reach of plant roots the system of irrigation is locally often referred to as "irrigation by percolation." It becomes a system when, as in the Mussel Slough country south of Kings River, the canals at the proper season supply the requisite amount of moisture to the cultivated tracts without burdening them with an injurious surplus during the rest of the year.

By the rising of the ground-water plane near Fresno large tracts of land have been permanently moistened. Some are very thoroughly subirrigated, and in many instances injuriously so. Relief must be sought in some localities either by drainage or by proper regulation of the supply of water to the district, whereby the elevation of the water table may be controlled.

Near the Fresno Canal, about 7 miles northeast of Fresno, lands were already too moist in 1882 to permit farming operations. Drain ditches 4 feet deep were then in use in Temperance Colony to drain the surface soil. In 1885 drain ditches 4 to 6 feet deep were in use in the Eisen vineyard, 5 miles east of Fresno, to keep ground water at least 4 feet below the surface of the ground. Parts of the town of Fresno, parts of Fresno and Central colonies, and lands to the north and west of Fresno may be cited as other instances of the too close proximity of ground water to the surface.

Some reference has already been made to the time of irrigating cer-

tain kinds of crops and the amount of irrigation accomplished by a stated amount of water. Some further illustration of the time when water is required for different products, and of the quantity absorbed by soils under the various conditions of irrigation in this vicinity, may not be out of place here.

Alfalfa, except when the water table is so near the surface that soil is kept moist from below during the entire growing season of this plant, is almost invariably irrigated once for each crop of hay cut. The first irrigation is early in spring and the first hay is cut in May; the succeeding ones at intervals of about five or six weeks. The number of crops cut is generally four; the average yield per acre is 1 to 1½ tons per cutting. Irrigations subsequent to the first of the season are given either before or just after the cutting of each crop. It is said in favor of irrigation before cutting (about one week or ten days) that then the ground's surface is shaded, it does not bake so hard in drying, and the new growth of alfalfa is better able to withstand the heat of the sun than if subjected to an irrigation after the hay is cured. But, whenever alfalfa is irrigated before cutting, in checks not perfectly level, water in some parts of these stands too long about the plant, too much of the plant is submerged, and the quality of the hay is impaired.

Where the ground water is so close to the surface that it supplies sufficient moisture to alfalfa land, occasional floodings are necessary to drown gophers.

Trees and vines were, until the last few years, very copiously and very frequently irrigated in this region. Trees were generally watered three or four times during the spring and summer; vines sometimes every two to three weeks. The water during the first years of irrigation near Fresno was, almost without exception, applied to the surface, and the result was that the roots of trees and vines frequently failed to penetrate into the subsoils, entailing evils which could not easily be corrected and which were particularly apparent in the quality of the product. The lesson was not quickly learned, but there has been decided improvement. Less moisture is supplied to the surface soils and greater care is taken to keep the surface well cultivated. Capillary attraction is thus interrupted at the depth to which the plow has penetrated, less moisture is brought into direct contact with the air, evaporation is consequently reduced, and plants are enabled to get the greatest possible benefit from the moisture already in the soil. Some examples from actual experience may illustrate how water is here used.

On June 16 to 17, 1882, 5 acres of alfalfa in small checks thoroughly leveled were irrigated in eight hours, from 6 p. m. to 2 a. m., in Central California Colony. The head of water used was 0.84 second-foot. The amount of water absorbed by the soil was equivalent to a depth of 0.11 foot over the entire 5 acres.

On the 17th of June, 1882, in Central California Colony, a supply of 2.26 second-feet was divided into four parts and each admitted into an

alfalfa check having an area of 1¼ acres. The irrigation of the alfalfa commenced at 6 a. m., and at 4 p. m. the water had been made to cover the entire area of each check. Alfalfa was being irrigated before cutting. The checks were level and no water was drawn off. The total amount supplied was equivalent to a depth over all of 0.37 foot. The soil was sandy loam; hardpan was 6 feet below the surface; ground water at about 12 feet. Less water would have been required if the whole stream had been admitted in turn to each check.

On June 12, 1882, in Central California Colony, two checks of alfalfa, 290 by 130 and 290 by 200 feet, respectively, were irrigated at the same time with 1.2 second-feet. Water was applied from 9 a. m. to 8 p. m. The total amount supplied was equivalent to a depth of 0.49 foot over the whole area of the alfalfa. The soil was a sandy loam, with hardpan at 1 to 2 feet. This alfalfa was to receive five such irrigations during the season.

On the Gould Ranch, north of Fresno, one attendant in 1882 handled about 1 second-foot of water. With this amount of water orchard lands were irrigated at the rate of about 4 to 5 acres per day of twenty-four hours. The average depth of the water applied over the whole surface of the orchard was 0.40 to 0.50 foot. The orchard received two to three irrigations per season. The trees of the orchard are set about 16 feet apart in rows 24 feet apart. Water was run in small ditches along each row, and was made to stand around the body of each tree while irrigation was in progress. The surface of the ground was not leveled when the orchard was set out. The soil is a heavy, dark loam, with hardpan at an average depth of 5 to 7 feet.

About 4 miles northeast of Selma 2 to 3 acres of vines or trees could be irrigated in 1882 in one day of twelve hours with 1 second-foot of water. The soil is a sandy loam, with no hardpan under it. It bakes on the surface in drying. Ground water was then some 40 feet below the surface. The method of irrigation is that of running water in furrows between rows. Several such irrigations per season are given to trees and vines.

In the same vicinity in 1882, irrigation of grain with 2 to 3 second-feet was progressing at the rate of 3 to 4 acres per day of twenty-four hours. The method of irrigation was the furrow method. Furrows were 10 to 12 feet apart. The soil was thoroughly saturated by percolation. The amount of water absorbed was sufficient to have covered the entire surface to a depth of 1.42 feet. One irrigation at any time after harvest is sufficient to mature the grain crop of the next season. The soil is a sandy loam, which bakes after wetting, unless cultivated before drying. Hardpan is not continuous; where it does occur it is generally at 8 feet below the surface. Ground water is at 35 to 40 feet.

On June 12, 1882, in Central California Colony a small check of grape vines, one-fourth acre, was irrigated in fifty-five minutes. The check was level and all water standing in it at the end of the fifty-five minute

was allowed to sink into the ground. The soil was still somewhat moist at the time this irrigation commenced. The soil is a sandy loam [illegible] feet deep, resting on a firm hardpan through which holes were [illegible] before the vines were set out. The amount of water supplied was [illegible] per foot, and the total amount was sufficient to water the ground [illegible] feet deep. The vines appeared thrifty and in excellent bearing. [illegible] irrigation was repeated about every three weeks until the grapes had matured.

About a mile east of Selma, where the soil is a sandy loam [illegible] feet deep, with ground water at [illegible] feet, it was said [illegible] acres of wheat land could be irrigated in one day of twenty-four hours [illegible] attendant with 1½ to 2 second-feet of water, equivalent to a depth of water over all of about 1.74 feet.

The foregoing statements must not be accepted as an indication of the ultimate duty of water on the Fresno plains. The area irrigated per unit of water will continue to increase, at least until irrigation produces no further permanent changes in the elevation of the ground water plane.

Without attempting to express in detail the acreage irrigated per cubic foot per second of canal capacity or actual flow of water, it may be noted that the total volume of water brought to the vicinity of Fresno during the seventeen years 1879 to 1895 would at a very low estimate have covered to an average depth of about 0.8 foot per annum the entire 260,000 acres of the region through which the canals send their branches. It would have covered the 50,000 acres to whose surface water is actually applied to an average depth of 4½ feet per annum, or to a total depth of 75 feet. Some of this water has of course been consumed in sustaining plant life, more has been evaporated, but the most of it still permeates the subsoils of the irrigated region and of the lands to the west.

WILDFLOWER.

The Wildflower country is a small portion of the east-side plain of San Joaquin Valley just south of the Kings River delta lands. Its limits may be considered coextensive with the area covered by the Emigrant Canal and its branches. Sandy soils predominate, the heaviest being near Cole Slough in the southeastern portion of this region, and the lightest along its northwestern border. The usual strip of bad lands, strongly alkaline, is found at the south, near the overflow limit from Kings River. Hardpan does not appear in a continuous layer under the soil; it is found in spots or belts of small extent, generally 4 to 8 feet below the surface, and usually but a few inches thick.

For a number of years all the water used for irrigation near Wildflower was supplied by the Emigrant Canal. As this canal became unreliable as a source of supply (due mainly to litigation concerning

ownership of its head works), water from other canals, the Fowler Switch and the Centerville & Kingsburg, was brought into the same district.

The irrigating period while the Emigrant Canal was in full service continued from March to the end of August. All the water of the canal during that period was apportioned to the distributing and irrigating ditches, and was allowed to sink from these into the subsoils. The method of irrigation was essentially that of subsurface saturation, which will be fully described in the notes on the Mussel Slough country. Alfalfa fields are sometimes subdivided into small rectangular checks of one-half acre to 2 acres each, and receive an occasional flooding. The surface of the country is very smooth, and no leveling or smoothing of the surface was necessary, as at Fresno. All water delivered into a check is allowed to sink into the soil, which readily takes it. Orchards and vineyards are irrigated, while water is available, by conducting it between the rows of trees and vines in small ditches. Summer crops are irrigated by the furrow method. Before lands were irrigated ground water was here at 10 to 16 feet below the surface; it has risen, as the result of irrigation, about 6 feet.

LIBERTY AND MILLRACE SETTLEMENTS.

The region watered by the Liberty and Millrace canals is a continuation of the Wildflower country westward, and the same notes with reference to soils apply. Water supply has usually been restricted to a few months each year, and the process of subsoil saturation has been rather slow. Irrigation is accomplished by flooding in small contour checks or by filling small ditches or furrows between rows of trees, vines, corn, or vegetables. Water is so freely absorbed by soils that no drainage is necessary.

LAGUNA DE TACHE RANCHO.

This grant, which extends southwesterly along the north side of the main channel of Kings River a distance in a direct line of about 25 miles, from near Kingsburg to Summit Lake, embraces the major portion of the north-side delta lands. Its hydrographic features have already been dwelt upon, and but little more is to be said concerning the use of water upon it for irrigation. The surface of the ground is smooth; there are no irregularities except such as have been produced by Kings River waters. Predominating soils are deep, sandy loams. Some of the depressions near present water courses, particularly in the northwestern and northern portions of the rancho, have sedimentary soil. The upper portion of the rancho, along Cole Slough, is in the alkaline belt. The surface slope from east to west is about 4 feet to the mile. Near Kings River is a fine growth of oaks. Willow thickets abound near the river in the southern portion of the rancho.

The many delta streams have always furnished an abundance of water to subsurface strata, so that depth to ground water has never been very great—4 to 16 feet, varying somewhat according to locality, and no doubt also somewhat with the seasons. The local control of fluctuations in the elevation of the water table is one of the features of the method of irrigation. The canal supply is ample from February to August. Little or no water is put upon the surface of the ground. Tracts to be irrigated have ditches generally located on the slight ridges built up by former water courses, from which water sinks into subsoils. Surplus waters are drained from the flat depressions between these ridges by a simple system of drain ditches. Crops irrigated are principally natural grasses and cereals.

MUSSEL SLOUGH COUNTRY.

Description.—All of the Kings River delta south of the main channel of the river and west of Cross Creek is commonly known as the Mussel Slough country. Many water courses, locally called sloughs, former channels of Kings River, course through this region from north to south. The heads of most of these have long been filled with deposits from the river. Others have more recently been artificially closed. A great many have been made parts of the distributing system of the irrigation works of this region.

The surface of the Mussel Slough country slopes from northeast to southwest at the average rate of about 4 feet to the mile, which is only about one-half the fall of the ground's surface in the vicinity of Fresno.

The soil of this region is all sedimentary. Most of it is sandy loam, varying somewhat in texture and color in different localities. It is sandiest toward the east and northeast; heaviest toward the west and south, where it merges into the peaty clays of the swamp-land belt. From northeast to southwest, at Grangeville, a strip of land several miles wide extends through this region, whose soil is a dark, rich loam, generally considered of the best in the delta. All soils are deep, resting upon subsurface strata of sands and soft, sandy clays.

Before the construction of the irrigation canals in the Mussel Slough country, depth to ground water was 10 to 18 feet. It was found at greatest depth in the eastern and southern portions of the delta. Irrigation very soon brought the water table close to the surface of the ground, and throughout the irrigated portion of the south-side delta it does not now sink to over 8 feet below the surface. It has an annual fluctuation, rising during the spring months when canals are full of water, and falling again in the fall of the year. In some localities this rise is so pronounced that standing water appears in the depressions, forming ponds which do not entirely disappear until the close of the irrigating season.

Canal system.—The Mussel Slough country is supplied with water

for irrigation from Kings and Kaweah rivers. The former sends wa through the Peoples, Last Chance, Lower Kings River, and Rhoa canals to the northern, central, and western portions of the south-si delta, while Kaweah River supplies water to its eastern portions throu the Settlers and Lakeside canals. For about six months each ye commencing with February, Kings River furnishes an abundance water for the canals leading from it. The canals from Kaweah Ri are not so copiously supplied with water, because that river is subj to greater irregularities of flow than Kings River. Their period flow is not so long as that of Kings River canals, and is larg dependent on the conditions of the diversion of water at the head of Johns River, where conflicting interests of different irrigated regi have given rise to repeated modifications of natural channels. Or narily the flow of Settlers and Lakeside canals commences in Janu and continues to the end of June.

The total amount of water supplied to the Mussel Slough coun by the canals above enumerated is sufficient to cover the entire a under these canals and their branches to an average of several f per annum. It was sufficient in 1885 to cover the land which was th in actual cultivation in this part of the Kings River delta to a dep of about 4 feet.

Irrigation on a noteworthy scale commenced in 1870 in the Mus Slough country when the Lower Kings River Canal was construct The construction and extension of the several other canals quick followed. There has been but little extension of the canal syst since 1880.

Each canal project (with the possible exception of the Mussel Slou Canal) had its origin with the settlers who needed water. The s consideration in each case seems to have been the securing of a fai reliable water supply at reasonable cost for use upon the lands of t projectors of each enterprise. This circumstance will account for t apparent duplication of canals; for, as the irrigated area was extend and additional lands were settled, it was but natural for the landown of the new settlements to construct new canals and to control in pendent canal works. The seven canals that are or have been in in the Mussel Slough country could well be replaced either by main canal from Kings River or, perhaps better, by one such canal one smaller one from Kaweah River.

The canals distribute water throughout the delta in a network branches and natural channels, often so intertwined that large ar are benefited by the water of several canals at the same time. happens, too, that lands whose owners have no claim upon water in canal are here and there so favorably located that they are irriga without cost to owners by the canals and ditches of their neighbors

Method of irrigation.—The rainfall in the Kings River delta is o *9 inches per annum.* Irrigation is necessary for the successful culti

of all soil products. Cereals and alfalfa were practically the only ıres from 1870 until 1884. More attention is now given to fruit ıng, to the cultivation of the grape, and to diversified farming.
› prevent wholesale inundation of south-side delta lands a levee is ntained along the south bank of the main channel of Kings River.

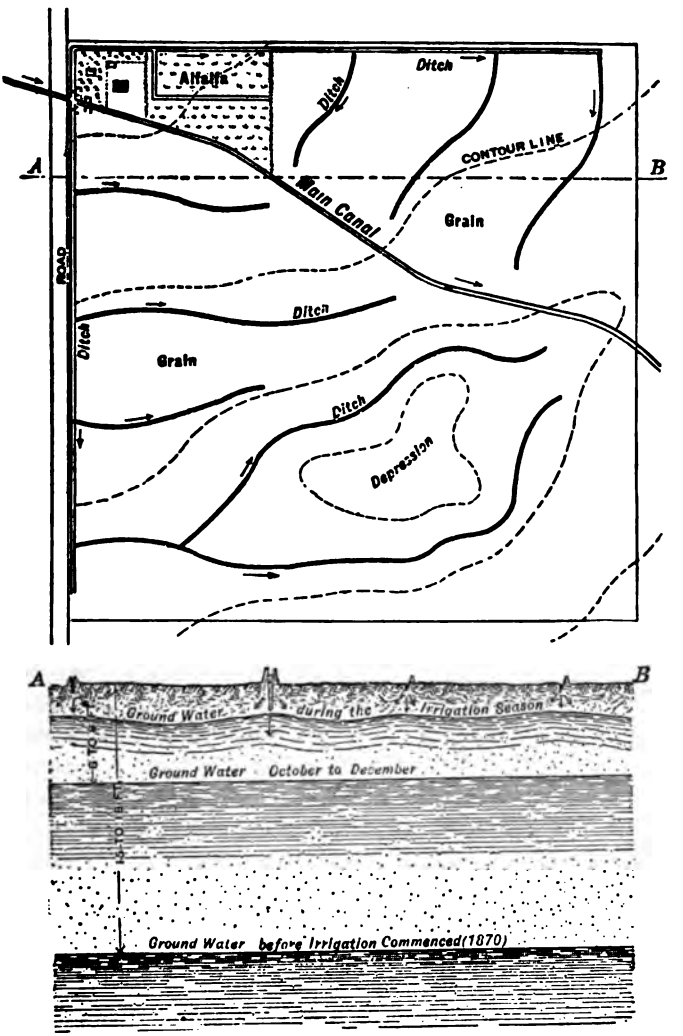


FIG. 21.—Irrigation by filling subsoils with water, Mussel Slough country.

ɜ inflow of irrigation water into the district is controlled by means ɜegulators in the heads of the principal canals. The flow of water ɜome of the main canals and in most canal branches is sluggish, ɜer being generally *held at a very light* grade, often by use of check ɜs.

Water is rarely spread over the surface of the ground. This is done only in seasons when the supply to the canals comes late, or when gophers become too numerous in the alfalfa fields. The ordinary method of irrigation may be described as a periodical saturation of subsoils. From the time that water becomes available in the spring the canals and their branches, as well as small irrigating ditches, are kept as full of water as the available supply will permit. The water which sinks into the porous soils gradually saturates subsurface strata, bringing ground water within reach of plant life and sometimes even out upon the surface. Irrigation is considered complete when the water table has risen to within 3 to 4 feet of the ground's surface, as this will insure a thorough moistening of surface soils by capillary attraction.

Throughout the Mussel Slough country the range of water-table elevation is clearly defined. It drops from its general position at the height of the irrigating season, about 4 feet below ground, to a greatest depth of 7 to 10 feet during the fall and winter months. This fluctuation is the direct result of irrigation, and it is more or less rapid in the different portions of the district according to the character of the soil, the distance between irrigating ditches, and other local conditions.

Locally this method of irrigation is rarely understood. It is referred to as irrigation by percolation, and according to the ordinary conception of the method there should be a direct lateral horizontal percolation of water through surface soils.

For the irrigation of grain or alfalfa it is customary, if the land be at all uneven on the surface, to carry distributing ditches along the highest points of the ground. From these the water is drawn into irrigating ditches 3 to 6 feet wide and 1 to 2 feet deep, which are cut with a slight fall out upon each side from the distributing ditches, and are from 300 to 600 feet apart. These irrigating ditches are either temporary or permanent. If temporary, they are usually plowed in after each irrigation and reopened for the next one. When orchards and vineyards are to be irrigated the ditches are constructed in straight lines between the rows of trees or vines, whenever such arrangement is possible.

Simple as this method of irrigation may appear, it has its disadvantages, not the least being directly due to the preponderance of the upward movement of the moisture in the surface soils. The water, before moving upward, often passes for long distances through permeable subsoils. Where ground is wet by irrigation 150 feet from the nearest ditch, the water reaching it must have passed at least 150 feet through porous strata, besides the distance it may have had to sink originally and the height it again rose to reach the surface soil. Water thus permeating, at times saturating, soils which are rich in salts ordinarily classed as alkaline, is sure to dissolve more or less of these mineral substances, and it will retain some of them in solution until sooner or later it arrives at the surface and there evaporates. Though the

quantity of these salts in solution may be very small, they will, if present at all, ultimately become apparent at the surface, because the moisture there evaporating is periodically replaced from below, not from above. Careful inquiry establishes the fact that some injury has already resulted from this cause; that many fields whose soil a few years ago was of excellent quality now have soil too strongly alkaline to produce good grain crops, and that even alfalfa fields have been injured. There seems to be no limit to the extent of damage which may result from this cause unless the method of irrigation be properly modified.

The simplest remedy for this evil, suggested by its cause, is to secure a preponderance of downward motion of moisture. If this can be done, more of the salts leached out of the soils by water will move downward away from the surface than upward toward it. The quantity of alkali at the surface, where it is most injurious, will necessarily be decreased. The remedy will be made still more effective if subdrainage be resorted to—if the water descending to the subsoils through the surface soil be in large part carried off in deep drainage channels.

It is not without interest in this connection to note the effect of rain in those spots of the plains where alkali is seen on the surface as an efflorescence or has formed a crust. A gentle rain, with no run-off from the surface, wetting the ground to the depth of a foot or two, may dissolve the efflorescence and all alkaline crusts, carrying the salts downward into the soil. But they return to the surface with the returning moisture, all of which is ultimately evaporated. A long-continued rain may send water vertically downward to ground water. It may thus cause a permanent reduction of alkaline salts at the surface. Sudden heavy rainfall, dissolving the salts on the surface, may carry them into natural drainage channels and permanently remove them from the soil.

It remains to be said that the accumulation of alkali at the surface is retarded by thorough tillage and occasional deep plowing. By tillage the capillary upward movement of moisture is checked; there is less evaporation from the surface layer of soil; consequently, the amount of alkali accumulating at the surface is minimized. Deep tillage buries the surface salts, removing them for a time at least from that layer of the soil where they are most injurious.

The cost of preparing land for irrigation in the Mussel Slough country has been relatively small because the surface required no shaping and only limited areas have been arranged for flooding in checks. The cost of constructing the small irrigating ditches has been variously estimated at from $2 to $4 per acre, and the annual expense of irrigating at from 25 cents to $1 per acre.

INDEX.

Page.

Alfalfa, methods of irrigating 83, 84, 90
Alta irrigation district, work in 52–57
Bacon & Crossmore Canal, description of .. 29–30
Bahwell Ditch, description of 33
Barton Ditch, description of 57
Bliss & Hyde Ditch, description of 25
Bostwick Cut, description of 33
Brown Ditch, description of 25
Burch Ditch, description of 32
Catron Ditch, description of 25
Cameron Creek, course of 13
Cameron Creek Company, operations of 29
Centerville Ditch, description of 47–48
Centerville & Kingsburg Canal, description of 50–52, 75–76
Chatten Ditch, description of 31
Cole Slough, course of 41–42
Consolidated Peoples Ditch, description of. 22–25
Crescent Canal, description of 62
Cross Creek, course of 13
Crowley & Mehrtens Ditch, description of .. 34
Curtis Ditch, description of 17
Curtis & Lindsey Ditch, description of 16
Cutler & Higgins Ditch, description of 31
Davis Ditch description of 25
Deep Creek course of 12, 14
Dennis & Jordan Ditch description of 57
Duty of water, observations on 79–85
Elbow Ditch No. 1, description of 16
Elbow Ditch No. 2, description of 16
Emigrant Canal, description of 57–58
Enterprise Ditch, description of 43
Evans Ditch, description of 32
Evans & Turner Ditch, description of 32
Evaporation and seepage, loss of water by .. 74–78
Extension Ditch, description of 25–26
Farmers Canal, description of 27–28
Fowler Switch Canal, description of 48–50
Fresno Canal, description of 43–47
Fresno Plains, canal system of 73–74
description of 71–73
effect of irrigation on water table in 78–79
loss of water by evaporation and seepage in 74–78
methods of irrigation and duty of water in 79–85
Fulgham Ditch, description of 33
Glenn Ditch, description of 57
Goshen Ditch, description of 20–21
Grant Canal, description of 58–62
Hall, W. H., cited 11, 39
Hamilton Ditch, description of 22

Page.

Hayes Lower Ditch, description of 21
Hayes Upper Ditch, description of 20–21
Hicks & Weston Canal, description of 20
Irrigation, effect on water table of 78–79
methods of 35–38, 79–85, 88–91
Jacobs Canal, description of 69
Jennings Ditch, description of 20
Judd Ditch, description of 30–31
Kaweah Canal, description of 26–27
Kaweah River, canals and ditches from 14–35
hydrography of 11–14
Kaweah and Mill Creek Water Company, operations of 31
Ketchum Cut, description of 34
Kincaid Ditch, description of 57
Kings River, canals and ditches from 42–69
course of 40–41
hydrography of 39–42
Kings River and Fresno Canal, description of 42–43, 74–75
Laguna de Tache Canal, description of 58
Laguna de Tache Rancho, irrigation on ... 58–59, 61, 86–87
Lakeside Ditch, description of 18–20
Last Chance Ditch, description of 65–67
Lee & Thompson Ditch, description of 25
Liberty Canal, description of 58
irrigation from 86
Longs Canal, description of 33
Lower Kings River Canal, description of .. 67–69
Matthew Ditch, description of 16–17
Mill Creek, course of 13
Mill Ditch, description of 33
Millrace Canal, description of 58
irrigation from 86
Modoc Canal, description of 20
Morrow Ditch, description of 57
Mussel Slough country, irrigation in 87–91
Mussel Slough Ditch, description of 64–65
Newell, F. H., letter of transmittal by 9
Oaks Ditch, description of 31–32
Ogden Ditch, description of 21–22
Orchards, methods of irrigating 83, 84, 86
Outside Creek, course of 14
Packwood Creek, course of 13
Packwood Creek Canal, description of 29–30
Pennebaker Ditch, description of 25
Peoples Ditch, description of 62–64
Pogues Lower Ditch, description of 21
Pogues Upper Ditch, description of 21
Reeds Ditch, description of 58
Rhoads Canal, description of 69

Page.
Rice Ditch, description of 25
Rocky Ford Ditch, description of 29
Rodgers Ditch, description of 20
St. Johns Channel, course of 12, 13
Seepage and evaporation, loss of water by .. 74–78
Selma irrigation district, situation of 52
Settlers Ditch, description of 17–18
Seventy-six Canal, description of 52–57
Soils, nature of 37, 72, 85, 87
Sunset irrigation district, work in 69–70
Swanson Ditch, description of 25
Sweem Ditch, description of 48
Teague Ditch, description of 25
Trees and vines, method of irrigating ... 83, 84, 86

Pag
Tulare irrigation district, work in 34–
Tulare Canal, description of 28–
Uphill Ditch, description of 1
Venice Ditch, description of 1
Vines and trees, methods of irrigating ... 83, 84, 8
Visalia Creek, course of 12, 1
Wahtoke Creek, course of 4
Wahtoke Reservoir, map of 5
Water, duty of 79–
Water table, effect of irrigation on 78–
Watson Ditch, description of 32–
Weston Ditch, description of .....
Wildflower, irrigation near 85–
Wutchumna Canal, description of 14–

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

RRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 19

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

J. C. Branner

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

GATION NEAR MERCED, CALIFORNIA

BY

CARL EWALD GRUNSKY

WASHINGTON

GOVERNMENT PRINTING OFFICE

1899

CONTENTS.

	Page.
etter of transmittal	7
ıa Joaquin River	9
Hydrography	9
Canals and ditches	13
Upper San Joaquin River Canal	13
Madera irrigation district	14
Chowchilla Canal	15
San Joaquin & Kings River Canal	18
Posey Slough or Dos Palos Canal	24
Temple Slough	24
Sanjon de Santa Rita	24
Blyth Canal	25
East Side or Stevenson-Mitchell Canal	25
resno River	26
Hydrography	26
Canals and ditches	27
Madera Canal (Fresno River Dam and Canal Company's Canal)	27
Adobe Ranch Ditch	31
ıowchilla River	31
Hydrography	31
Canals	31
Kohler, West & Minturn canal system	31
Buchanan Reservoir project	32
erced River	33
Hydrography	33
Canals and ditches	34
Merced or Crocker-Huffman Canal	34
Merced River Bottom canals	37
Merced Falls mills	37
Merced Falls Ditch	37
Murray Mill Ditch	37
Snelling Ditch	38
Montgomery & Anderson Ditch	38
Ingalsbe Slough	38
Scott Ditch	38
Dean Ditch	38
Old River	38
Feldhaus & Reed Ditch	39
Hamlin or Millrace Ditch	39
Griffith & Shaver Ditch	39
Methods of water diversion and use	39
olumne River	40
Hydrography	40
Canals and ditches	43
Lagrange Ditch	43
Wheaton Ditch	44
Lagrange Dam	45

Tuolumne River—Continued. Page.
Canals and ditches—Continued.
Turlock irrigation district 47
Modesto irrigation district 49
Stanislaus River 50
Hydrography 50
Canals and ditches 52
Stanislaus & San Joaquin Canal 52
Knights Ferry & Table Mountain Ditch (Pentland Ditch) 55
Oakdale Canal 56
Mokelumne River 56
Hydrography 56
Canals and projects 57
West-side creeks and other streams of San Joaquin Valley 58
Index 59

ILLUSTRATIONS.

PLATE I. Steamboat chute, San Joaquin River, at head of San Joaquin & Kings River Canal: *A*, As seen from below; *B*, As seen from above 20
II. Regulator in San Joaquin & Kings River Canal: *A*, As seen from above; *B*, As seen from below 22
III. Blyth Canal: *A*, Head at San Joaquin River; *B*, Nine-foot cut 24
IV. East side of San Joaquin Valley from Chowchilla River to Merced River 32
V. Dam across Merced River for Crocker-Huffman Canal 34
VI. Regulator in Crocker-Huffman Canal 34
VII. Tunnel No. 2 on Crocker-Huffman Canal 36
VIII. Crocker-Huffman Canal near head, looking down 36
IX. Inlet to Merced Reservoir, Crocker-Huffman Canal 38
X. Lagrange Dam during construction: *A*, As seen from front; *B*, Abutment 44
XI. Lagrange Dam: *A*, Nearing completion; *B*, At time of high water 46
FIG. 1. Discharge of San Joaquin River at Herndon, California, 1895–96 12
2. Head of Upper San Joaquin River Canal, on San Joaquin River, 1883. 13
3. Plan and section of Chowchilla Canal regulator 16
4. Head of San Joaquin & Kings River Canal 18
5. Details of head works of San Joaquin & Kings River Canal 19
6. Brush dam across San Joaquin River at head of San Joaquin & Kings River Canal 20
7. Head of Fresno River Canal 27
8. Head of Adobe Slough 30
9. Water diversion by Kohler, West & Minturn Canal 32
10. Discharge of Tuolumne River at Modesto, California, 1895–96 42
11. Discharge of Tuolumne River at Lagrange, California, 1895–96 43
12. Plan of Lagrange Dam 45
13. Section of Lagrange Dam 46
14. Discharge of Stanislaus River at Oakdale, California, 1895–96 52

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, July 15, 1898.

IR: I have the honor to transmit herewith a manuscript entitled gation near Merced, California, by Mr. C. E. Grunsky, and to recom- d that it be published as one of the series of pamphlets on Water ply and Irrigation. This is the third of three papers relating to gation in San Joaquin Valley, the other two being Nos. 17 and 18 of series. This paper describes the irrigation systems deriving their er supply from San Joaquin, Chowchilla, Merced, Tuolumne, Stan- us, and Mokelumne rivers.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

lon. CHARLES D. WALCOTT,
Director United States Geological Survey.

IRRIGATION NEAR MERCED, CALIFORNIA.[1]

By C. E. GRUNSKY.

SAN JOAQUIN RIVER.

HYDROGRAPHY.

San Joaquin River enters San Joaquin Valley from the east at a int about 20 miles northeast of Fresno, flows southwesterly 55 miles the valley trough, and, entering the lowlands of the Fresno Slough untry, turns at Las Juntas abruptly to the northwest and becomes e main drainage channel of the valley. For 40 miles below Hamptonlle, the point at which the river enters the main valley, its course s through narrow strips of bottom land hemmed in by bluffs rising the plains above. These bluffs are generally one-half mile to a mile part and descend abruptly toward the river from a very smoothrfaced plain. They gradually decrease in height toward the west, eir height being about 60 feet near the eastern edge of the valley and feet near Herndon. Before the river reaches Las Juntas, 20 miles y river below Herndon, the bluffs have disappeared entirely, and the ood waters of the river sweep across country to the southwest into resno Swamp and to the northwest in a direction down the valley arallel with the river.

Bottom lands of small extent flank the river between its bluffs, but ave remained unirrigated. The upland of the main valley plain is so gh above the river near the eastern border of the valley that canals command the upland would be very costly, and none have been conructed, though several have at various times been projected. The version of water for irrigation is confined to that portion of the river ing in or near the main valley trough.

The area of the river drainage basin above the point where it breaks m the Sierra Nevada is 1,637 square miles. This entire area is ountainous. It extends eastward to the summit of the Sierra Nevada, miles of this summit, measured in a direct line, being the eastern nit of the river's drainage basin.

The river is ordinarily at its highest stage in the spring months, genally in May and June; but winter freshets are not uncommon, and treme floods, such as those of 1861–62 and 1867–68, have occurred in nter. The flow of the river at Hamptonville, near the eastern border San Joaquin Valley, was approximately determined by the State

[1] *See introductory remarks in* Water-Supply Paper No. 17, p. 13.

engineer of California for the years 1878 to 1884.[1] A mean monthly flow for November of 378 second-feet and a mean monthly flow for June of 10,150 second-feet were the extremes indicated by these measurements.

The river is ordinarily at a low stage from September to the end of December. It is at a medium stage, with a flow in excess of 1,000 second-feet, from January to March and in July and August. It is high from April to July. These periods are, however, to be taken only as a general guide, because the flow of this stream varies within wide limits, according to the varying conditions of rainfall and temperature of each season.

The Southern Pacific Company have maintained daily readings of gage height on this river since 1879 at their bridge half a mile north of Herndon and 12 miles north of Fresno. Since 1895 the United States Geological Survey has been taking measurements of discharge from the wagon bridge near by. The results of these measurements are published in various reports of the Survey.[2]

The following facts in regard to the flow of the river are brought out:

1895. Maximum discharge, May, 19,960 second-feet; minimum, September, October, November, and December, 260 second-feet; average for the year, 3,791 second-feet.

1896. Maximum discharge, May, 18,800 second-feet; minimum, September and October, 80 second-feet; average for the year, 2,756 second-feet.

1897. Maximum discharge, May 24, 18,600 second-feet; minimum, December 23, 60 second-feet; average, 3,105 second-feet.

Estimated monthly discharge of San Joaquin River at Herndon, California.[3]

[Drainage area, 1,637 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1895.						
January	11,225	1,260	2,881	177,102	2.029	1.760
February	8,500	909	2,568	142,590	1.634	1.569
March	9,318	1,554	2,779	170,840	1.957	1.697
April	9,863	3,354	5,834	316,600	3.626	3.564
May	19,960	5,100	13,124	807,000	9.243	8.017
June	14,565	7,250	10,674	599,404	6.866	6.520
July	8,000	2,148	4,528	278,410	3.189	2.766
August	2,301	793	1,417	87,104	.997	.866
September	8,500	260	1,085	64,567	.740	.663
October	1,260	260	420	25,846	.296	.257
November	426	260	362	25,562	.247	.221
December	677	260	373	22,905	.262	.228
The year	19,960	260	3,791	2,717,930	31.086	2.334

[1] See Physical Data and Statistics, Wm. Ham. Hall, State engineer of California, 1886, pp. 449, 476.

[2] Twelfth Ann. Rept., Part II, 1891, p. 321, Pl. LXXXIV; Bull. No. 131, 1895, p. 81; Bull. No. 140, 1896, p. 290; Eighteenth Ann. Rept., Part IV, 1897, p. 385; Water-Supply and Irrigation Paper No. 11, 1897, p. 91; Water-Supply and Irrigation Paper No. 16, 1898, p. 190.

[3] In using results shown in these tables of discharge, some allowance must be made for uncertainty as to the permanency of stream cross sections on which rating tables were based. A slight change in absolute elevation of river bed at the rating station, if known, would modify the rating table, particularly for low-water stages. Lack of information on this point may lead to erroneous conclusions. *The flow of San Joaquin River at Herndon* has probably never dropped to 60 cubic feet per second.

Estimated monthly discharge of San Joaquin River at Herndon, California—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1896.						
January	12,800	250	2,119	130,335	1.49	1.29
February	1,750	950	1,177	67,696	.77	.72
March	12,176	1,180	2,612	160,649	1.84	1.60
April	5,648	1,920	2,675	159,192	1.83	1.64
May	18,800	2,560	5,394	331,700	3.81	3.30
June	16,920	6,544	11,799	702,106	8.00	7.21
July	8,080	1,600	4,177	256,865	3.04	2.55
August	1,300	700	1,048	64,463	.74	.64
September	1,180	80	534	31,817	.37	.33
October	700	80	167	10,275	.12	.10
November	2,390	430	697	41,492	.48	.43
December	1,750	430	666	40,957	.47	.41
The year	18,800	80	2,756	1,997,547	22.96	1.69
1897.						
January	1,180	535	655	40,275	.46	.40
February	15,660	1,180	2,598	144,286	1.66	1.59
March	4,600	1,675	2,325	142,960	1.64	1.42
April	11,708	2,305	6,541	389,216	4.46	4.00
May	18,600	9,565	13,545	832,855	9.54	8.27
June	12,332	2,390	5,862	348,812	3.98	3.57
July	4,300	1,300	2,493	153,290	1.75	1.52
August	1,300	590	898	55,216	.63	.55
September	535	105	227	13,507	.16	.14
October	820	105	279	17,155	.20	.17
November	4,300	120	872	51,887	.59	.53
December	4,000	60	968	59,520	.68	.59
The year	18,600	60	3,105	2,248,979	25.75	1.90

For several miles below the western base of the Sierra Nevada foothills ledges of rotten granite appear in the bed of the river; these are often covered to considerable depth by sand and gravel bars. The river channel is here generally 300 to 500 feet wide. From Hamptonville to Herndon it falls 89 feet in 20 miles; thence to Las Juntas about 36 feet in a distance of about 36 miles. Below Herndon the low-water flow is between sandbars. After passing Las Juntas, where the river is joined by Fresno Slough, it becomes the main drainage line of San Joaquin Valley. It has not, however, been a rapid land builder, and the elevation of river banks is but slightly greater than that of the general surface of the submersible area which flanks the stream.

From Las Juntas to the head of Old River, where the river bifurcates, the distance in an air line is 87 miles. The river between these points has a length of 146 miles. Its average width is 400 feet, and the depth of its channel, from top of natural banks to river bed, is 12 to 18 feet. There is a slight slope of bank lands away from the river, except at a few points, where the river impinges upon spurs of upland that extend eastward from the Coast Range foothills, notably at Grayson and near San Joaquin. The river bottom is generally clean sand, but a firm clay is here and there exposed. River banks are of quite firm alluvial material, showing but little tendency to cave. They are lightly fringed with willows and cottonwoods and occasionally oaks.

The land upon either side of the river subject to periodical flooding

is generally open country, throughout which clumps of willows mark the alignment of sloughs, as the overflow channels from the river are called. Tule grows in spots where high bank lands along the sloughs or river have penned in water or sufficiently retarded drainage to keep the surface moist throughout the year. Oaks are dotted over some portions of this region, but rarely appear in dense groves. The main east-side and west-side plains have been described in Water-Supply Paper No. 17, on page 15.

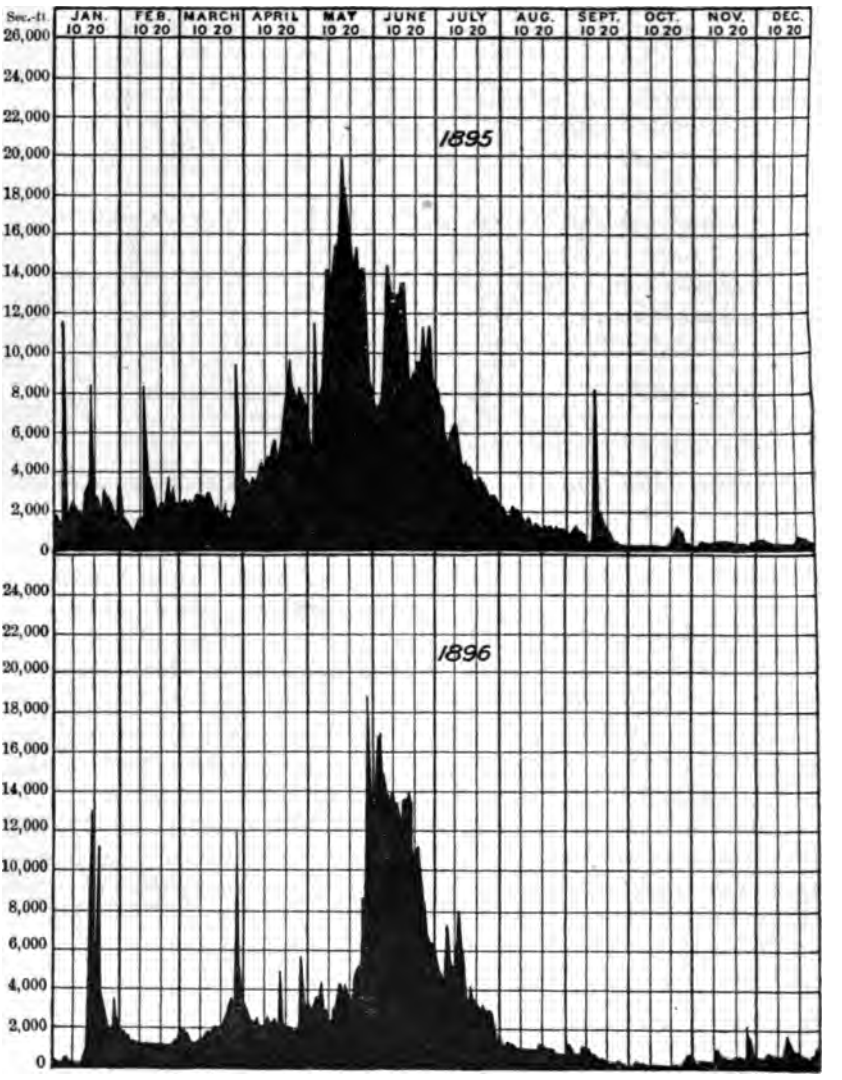


FIG. 1.—Discharge of San Joaquin River at Herndon, California, 1895–96.

CANALS AND DITCHES.

pper San Joaquin River Canal.—This work has not been a success. construction of the canal commenced in 1879. It was pushed for e time, until about $85,000 had been expended on it; several years ı passed, during which little or no work was done. A new com- y finally took hold of the enterprise, expending about $300,000 on a ı at the head of the canal and on the main canal above Herndon. ir dam was completed in the fall of 1883 and was maintained

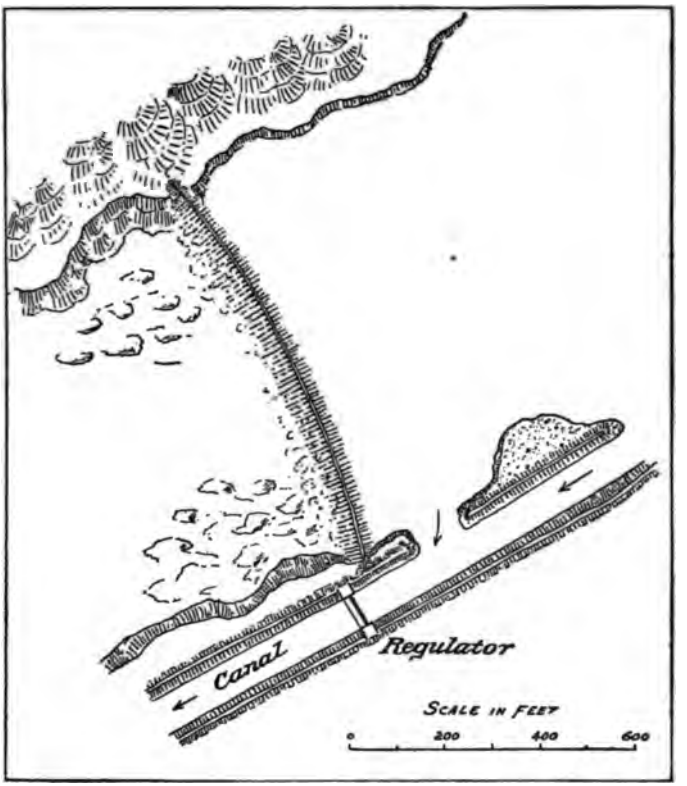


FIG. 2. —Head of Upper San Joaquin River Canal, on San Joaquin River, 1883.

ı difficulty for several years, during which time a little water was ɜd through the canal and out upon the upland near Herndon. ut 1887 the work was abandoned.

ıe head of the canal is about one-half mile below Hamptonville, at a t where the river is nearly 900 feet wide. The river bottom is sand, ıt 20 feet deep, resting upon a rotten granite formation. A loose-rock rting dam was constructed at this point to a height of about 8 above low water. It was proposed to allow the river to flow over from bank to bank. The material used for the dam was rock as *ried from a spur of granite,* cobbles, and gravel. All material was

dumped into place from small cars. No attempt seems to have been made to secure large blocks of stone, and all rock fragments ordinarily classed as quarry waste went into the structure. The largest stone (maximum size about 1¼ cubic yards) was dumped downstream to form a face that should resist the force of overfalling water. As the lower face rock dropped into place the river cut out the sand in its bed, thus permitting at least the toe of the structure to find a solid formation. The upstream face was formed of fine quarry waste, cobbles, and gravel. Trestle piles which carried the car track were allowed to remain in place and the downstream crest edge of the structure was protected by three layers of planking. Other expedients, such as extending an iron plate over the upper portion of the downstream slope for its protection, were tried, but experience soon demonstrated the difficulty of maintaining a loose-stone dam in a river of considerable flow. Even the later reconstruction of the downstream face of the dam with large blocks of stone, hand laid, and its partial protection with fascines could not make the structure permanently secure and the river's freshets repeatedly cut out sections of it.

Experience with the canal itself was equally unfortunate. Its course for several miles is along and across the river bottoms, thence for 9 miles it hangs on the steep descent or bluff, from south-side upland to river, and finally cuts out to the higher level of the upland a few miles above Herndon. Near its head the canal has a bed width of 75 feet. Its total fall in the first 4 miles does not exceed 1 foot. Along the bluff it received somewhat irregular dimensions. It was intended to carry water 5 feet deep. Its bed width was generally 24 feet, side slopes 3 to 1, and its gradient one-quarter to 1 foot fall to the mile. It was endeavored to make sidehill cutting supply material for an outer levee, whose slope away from the canal was the natural slope of the material used in its construction, generally between 1½ to 1 and 2 to 1. Some of the excavated material was removed by blasting, being in firm layers of very hard clay or ashy sandstone. These firm materials, alternated with soft ashy deposits or a reddish loam material, did not make a safe levee, and great trouble was experienced in making this levee hold water.

Water was furnished through the canal long enough to encourage some tree planting and diversified farming, but not much progress had been made when the canal fell into disuse.

Madera irrigation district.—This irrigation district is located on the east-side plain of San Joaquin Valley, to the north of San Joaquin River. The district was organized in 1888 and was to be supplied with water from San Joaquin River. Notwithstanding active opposition, the necessary bonds were voted and the legality of the district organization was affirmed by the courts. It was proposed to take water from the north side of San Joaquin River near the point where it leaves the hills, but no construction work was done. Operations were blocked

by legal proceedings, and after many long delays it was decided to disorganize, which has been done. The district had an area of 305,000 acres, and the amount of bonds voted for construction of works was $850,000.

Chowchilla Canal.—This canal, which was constructed in 1872, has its head on the north side of San Joaquin River about 8 miles by river above Las Juntas. Its course is slightly west of north, almost parallel with the main drainage of the valley, and for a distance of over 20 miles it commands a strip of land averaging 5 miles in width, which is bordered on the west by San Joaquin River.

The river at the head of the canal lies in several channels, one of which sends off a branch toward the north, known as Lone Willow Slough. The river banks at the head of this slough rise about 15 feet above the low-water plane. By deepening the slough and constructing a gate in it, near its head, it was converted into a section of the canal, and from it, about 2 miles below, water was turned into the artificial canal. The canal was made 30 feet wide on the bottom and 2½ feet deep near the point where it leaves Lone Willow Slough. But these dimensions are maintained for 3 to 4 miles only. The width is then gradually reduced to 25, to 20, and finally to 15 feet. In its northerly course of nearly 30 miles the canal follows a grade line having a fall of 6 inches to the mile.

Mr. J. D. Schuyler, as assistant State engineer in 1879, describes the difficulties of canal construction and the canal head gate as follows:[1]

The total cost of the canal was, in round numbers, $100,000.[2] The tough character of the excavation, the greater portion of the distance rendering the free use of blasting powder necessary, greatly increased the cost over that of ordinary earthen channels of like dimensions.

The treacherous character of the quicksand bed at the head of the canal has made it an exceedingly difficult task to construct a head gate that would withstand the action of floods. For four successive seasons after the building of the canal the head gate was washed out, and as often replaced by a new structure on a different plan.

In the fall of 1877 the present structure, which has withstood severe tests, and gives every evidence of permanence, was built. As this structure has some novel features, and its plan is indorsed by its success, it deserves a brief description. The use of sheet piling, generally considered indispensable to the safety of structures of this kind, was in this instance abandoned. The foundation was prepared by spreading a layer of loose sandstone, one or two feet thick, evenly laid on the quicksand. Upon this a floor of 2-inch plank, without sills, was laid across the axis of the canal, to which was spiked another layer of plank laid lengthways. This floor was placed 6 feet below the level of the bed of the river, and upon it was built the structure of 10 by 10 timbers, having a length of 60 feet, a width of 3 feet, and a height of 10 feet of clear waterway. The whole was weighted with an embankment of earth, 10 feet high, placed on top of the structure, the weight exceeding 500 tons. The sluiceways or regulating gates were placed 5 feet from the upper end of the floor, the space of 5 feet in front being occupied by a box of heavy timbers filled with rock and floored over, the top being 1 foot below the bed of the

[1] See Report of State Engineer, 1880, Wm. Ham. Hall, Part IV, Appendix, p. 176.

[2] This is believed to be an over estimate. The original total cost of the main canal was probably nearer $50,000 than $100,000.

river. The water comes into the canal therefore with a free overfall of 5 feet. In lieu of heavy solid regulating gates which are difficult to raise and lower, loose planks 5 feet long placed in grooves in the vertical posts are used. They are removed by a double hook which engages on an iron pin passing through the plank and projecting two or three inches on each side. A rope and windlass assists in raising the planks to the top. The cost of this structure was $3,200, of which $2,000 was expended in laying the foundation.

There is no weir or dam in the river to aid in accomplishing diversion of water into the canal, but the canal bed is below the low-water plane of the river (reported to be 6 feet below), affording a fall from river to canal which insures it a full supply of water.

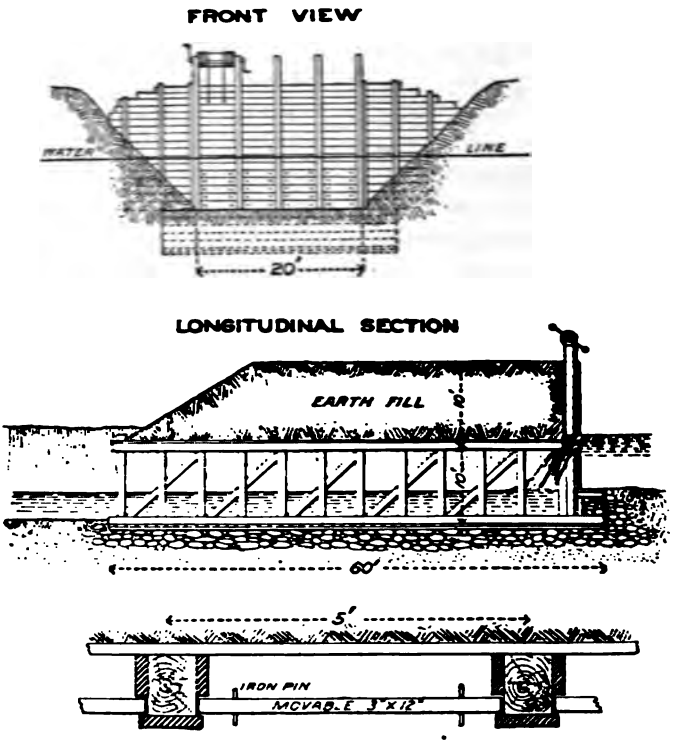


FIG. 3.—Plan and section of Chowchilla Canal regulator.

The head gate above described is located in Lone Willow Slough, about 100 feet from the river bank. A second regulator is maintained in the head of the artificial canal, and the lower portion of Lone Willow Slough is used as a channel for waste water.

The canal crosses the natural drainage of the east-side plain, a location which has at times made maintenance and repair expensive, owing to the frequent destruction of canal banks by the water reaching the canal from the east. At the crossings of Cottonwood Creek, Fresno River, Berenda Slough, and Ash Slough the canal is left open toward

the east, and an outlet gate in its west or lower bank permits the passage of the creek waters when they are swollen by freshets.

The canal is owned jointly by the Miller & Lux Company and the California Agricultural and Pastoral Association. The interests of the two concerns are equal to the point where the canal leaves the lands owned by the former; below that point the latter is sole owner.

The canal capacity near its head is from 100 to 200 second-feet. Its water is now used largely for the irrigation of alfalfa, although when first undertaken the work was intended principally for stock purposes. The place of use is on the Riverside and Columbia ranches of the Miller & Lux Company and on the Chowchilla Ranch of the California Pastoral and Agricultural Association. No water is sold from this canal, it being all required for use on the three tracts named.

The total area of alfalfa land now under irrigation is about 7,000 acres, nearly equally distributed to the three tracts named. On the Chowchilla Ranch works have recently been constructed for extensive irrigation of grass lands. These consist of high well-built levees extending from the high bank land of San Joaquin River eastward far out upon the plain. These levees, six in all, are about a mile apart, and each is equipped with a large so-called "stop gate." These gates are similar to the check weirs in ordinary use on large canals. The largest are 40 feet wide, and some of these are reported to have cost $1,500 each. Each of these gate structures supports a bridge. They are used to restrain water until the submersible area above each is covered with water, which then flows through the gate to the next tract to be inundated. For this grass-land irrigation the flow of the canal and the natural flow of the streams intercepted by it supplement the flow of the Blyth Canal, hereinafter referred to.

The method of irrigating alfalfa lands is by flooding in contour checks. The check levees on the Chowchilla Ranch were originally made high and had exceptionally steep side slopes. The areas of the compartments or checks ranged from 20 to 100 acres. They were altogether too large for the available water supply. It was at first attempted to make the gates through the check levees serve as roadways for teams and farming implements, but this idea soon proved impracticable. The advantage of reducing check areas and of making low flat levees has long been recognized by the managers of the property, and the original defects in the irrigation system have in a measure been remedied.[1] The Riverside Ranch receives its water through a branch canal which was constructed in 1884. This branch is taken out of Lone Tree Slough about 1½ miles below its head and closely follows the river. It has a length of about 8 miles. The soil on the Riverside and Columbia ranches is a heavy black loam. The original practice was to subdivide into irregular checks of 10 to 50 acres by means of high, steep levees. It was found necessary to make use of natural sloughs and waterways to carry off

[1] See Report of State Engineer, Wm. Ham. Hall, 1880, Part IV, Appendix, p. 177.

IRRIGATION NEAR MERCED, CALIFORNIA.[1]

By C. E. Grunsky.

SAN JOAQUIN RIVER.

HYDROGRAPHY.

San Joaquin River enters San Joaquin Valley from the east at a nt about 20 miles northeast of Fresno, flows southwesterly 55 miles the valley trough, and, entering the lowlands of the Fresno Slough ntry, turns at Las Juntas abruptly to the northwest and becomes e main drainage channel of the valley. For 40 miles below Hamptonle, the point at which the river enters the main valley, its course s through narrow strips of bottom land hemmed in by bluffs rising the plains above. These bluffs are generally one-half mile to a mile part and descend abruptly toward the river from a very smoothrfaced plain. They gradually decrease in height toward the west, eir height being about 60 feet near the eastern edge of the valley and feet near Herndon. Before the river reaches Las Juntas, 20 miles river below Herndon, the bluffs have disappeared entirely, and the ood waters of the river sweep across country to the southwest into esno Swamp and to the northwest in a direction down the valley rallel with the river.

Bottom lands of small extent flank the river between its bluffs, but ve remained unirrigated. The upland of the main valley plain is so gh above the river near the eastern border of the valley that canals command the upland would be very costly, and none have been conucted, though several have at various times been projected. The ersion of water for irrigation is confined to that portion of the river ng in or near the main valley trough.

The area of the river drainage basin above the point where it breaks m the Sierra Nevada is 1,637 square miles. This entire area is untainous. It extends eastward to the summit of the Sierra Nevada, miles of this summit, measured in a direct line, being the eastern it of the river's drainage basin.

The river is ordinarily at its highest stage in the spring months, genlly in May and June; but winter freshets are not uncommon, and reme floods, such as those of 1861–62 and 1867–68, have occurred in nter. The flow of the river at Hamptonville, near the eastern border San Joaquin Valley, was approximately determined by the State

[1] *See introductory remarks in* Water-Supply Paper No. 17, p. 13.

IRRIGATION NEAR MERCED, CALIFORNIA.[1]

By C. E. GRUNSKY.

SAN JOAQUIN RIVER.

HYDROGRAPHY.

n Joaquin River enters San Joaquin Valley from the east at a about 20 miles northeast of Fresno, flows southwesterly 55 miles e valley trough, and, entering the lowlands of the Fresno Slough ry, turns at Las Juntas abruptly to the northwest and becomes ain drainage channel of the valley. For 40 miles below Hampton- the point at which the river enters the main valley, its course hrough narrow strips of bottom land hemmed in by bluffs rising e plains above. These bluffs are generally one-half mile to a mile and descend abruptly toward the river from a very smooth- ced plain. They gradually decrease in height toward the west, height being about 60 feet near the eastern edge of the valley and t near Herndon. Before the river reaches Las Juntas, 20 miles er below Herndon, the bluffs have disappeared entirely, and the waters of the river sweep across country to the southwest into o Swamp and to the northwest in a direction down the valley lel with the river.

tom lands of small extent flank the river between its bluffs, but remained unirrigated. The upland of the main valley plain is so above the river near the eastern border of the valley that canals nmand the upland would be very costly, and none have been con- ed, though several have at various times been projected. The sion of water for irrigation is confined to that portion of the river in or near the main valley trough.

e area of the river drainage basin above the point where it breaks the Sierra Nevada is 1,637 square miles. This entire area is tainous. It extends eastward to the summit of the Sierra Nevada, les of this summit, measured in a direct line, being the eastern of the river's drainage basin.

e river is ordinarily at its highest stage in the spring months, gen- in May and June; but winter freshets are not uncommon, and ne floods, such as those of 1861–62 and 1867–68, have occurred in r. The flow of the river at Hamptonville, near the eastern border n Joaquin Valley, was approximately determined by the State

[1] *See introductory remarks in* Water-Supply Paper No. 17, p. 13.

to be used. They pay for water by the acre irrigated. The canal company reserves the right to charge for water wasted, when such waste can be ascertained. When there is a shortage of supply, the canal superintendent distributes water to the irrigators in proportion to the acreage for which water is required. If an insufficient quantity of water is delivered to effect the proposed irrigation, a corresponding reduction in the water bill is made. Charges for water are made by the acre, according to character of crop irrigated. The rates in force preceding 1880 were about as follows:[1]

For cereals during any part of the season from July 1 of one year to the same date the following year, $2.50 per acre.

For alfalfa, $3 per acre per year.

For market gardens, $5 per acre per year.

For wild-grass lands, 75 cents per acre per year.

For the irrigation of a second crop during the season, $1.

For the fall irrigation of lands, from July 1 to January 1, the charge for a single application is $1.50 per acre. After January 1, any number of waterings necessary to mature a crop of cereals is given for $1.

These rates have at various times been modified somewhat. The rate for alfalfa has recently been $2.50 per acre per year, and, under agreement to take water for the season, application being made before October 1, $2 per acre. No charge is made for the irrigation of small gardens of those irrigating large tracts of grain or alfalfa. The special contract made with Messrs. Miller & Lux gave them water at about one-half of the above rates.

The distribution of water from the canal to irrigators is in charge of a canal superintendent. Those requiring water must give ten days' notice and must specify the number of acres for which water is wanted, as well as the particular tract of land to be irrigated.

The soils to which water is applied from this canal are thus described by Mr. James D. Schuyler:[2]

The lands irrigated by the San Joaquin and Kings River Canal vary so greatly in character that the practice of irrigation is not uniform. From the point where irrigation first begins, near Firebaugh's, 8 miles below the head of the canal, to its terminus at Orestimba Creek, there is a gradual change in the quality, depth, and texture of the soil, from the minimum of absorbtiveness and the consequent maximum number of irrigations required to produce crops, to the maximum of absorbtiveness and the minimum of applications necessary. The soils which retain moisture longest are those which absorb most water and are consequently best adapted for irrigation. In no irrigated section of the State are the extremes in quality of soil more marked. The adobe soils do not extend the entire distance from the canal to the river, but seem to be principally limited to a strip 2 to 5 miles wide from Firebaugh's to Los Baños Creek. They are shallow—from 1 to 2 feet deep—and underlaid with yellow clay hardpan. East of this strip to the river the soil is generally of an alluvial character, highly susceptible of profitable irrigation, but not heretofore provided with facilities for irrigation, the land being devoted exclusively to grazing purposes. On either side of Los Baños Creek the soil is a

[1] Report of State Engineer, 1880, Part IV, Appendix, J. D. Schuyler.
[2] Idem, p. 166.

A.

B.

REGULATOR IN SAN JOAQUIN & KINGS RIVER CANAL

A. As seen from above. *B.* As seen from below.

ssible. This was done with excellent success. The canal bed was ide 45 feet wide, with its sides practically vertical, and the water pth was increased to 6 feet. The soil of the banks, impregnated th the mud deposits from the canal water and rendered resistant by e roots of plants which had penetrated it in all directions, showed further tendency to erosion, and the great inshore depth of water, as ell as its velocity, retarded the growth of plants.

Below Los Baños Creek the fall of the canal was made only 6 inches the mile. This 6-inch fall practically commences 2 miles above Los años Creek, as for some reason the grade line of the canal bed was ised 1 foot at that point. Experience on this canal seems to justify e conclusion that for a canal of its size a slope as light as 6 inches to e mile may safely be adopted.

This canal crosses the natural drainage lines of the west-side plain right angles. The water courses are torrential in character. They ve no flow in years of scant rainfall, and in the occasional wet winters ey respond quickly to the local storms, but do not maintain their flow ng. Where crossed by the canal the arrangement of the crossing is tremely simple. A check weir is put into the canal just above and st below the creek channel, and an embankment, usually of gravel, carried across the creek channel just below the canal and connected ith the canal levee. When the creek rises and overtops this barrier, is quickly cut out by the water, in whole or in part, and upon subdence of the freshet is replaced. The flow of the creek, if there be y after the freshet, is received into the canal.

The San Joaquin & Kings River Canal was intended to be used for vigation as well as irrigation purposes, and in accordance with that ea the structures originally planned for it included arrangements for ssing canal boats. They were all timber structures of unnecessarily avy and cumbersome design and correspondingly expensive. The ore modern structures on the canal are of the usual light inexpensive pe so common on the canals of the West.

The distributing ditches, which usually lead off from the main canal arly at right angles, are generally 8 to 12 feet wide on the bottom d carry water 2 to 3 feet deep. Their aggregate length is estimated about 200 miles.

The cost of the canal and works connected therewith has been about ,000,000, not including about $300,000 expended in maintenance and pairs. The dividends paid have probably not exceeded $250,000. At e time of this writing there is a supplementary high-line canal being nstructed which will greatly increase the capacity and efficiency of is canal system. This new canal leaves the original canal about 3 iles below its head and gradually separates from it, being carried on grade of only 6 inches to the mile. It has a bed width of about 45 et and is generally about 1 mile distant from the main canal, with hich it is connected by means of a number of short branch ditches.

Posey Slough or Dos Palos Canal.—This is a work which hardly deserves enumeration among irrigation canals, as it amounts to but little more than a control of the inflow into a natural high-water channel which breaks out of San Joaquin River about 8 miles below Firebaugh's. It was necessary in protecting west-side lands against overflow to either close the slough with an embankment connected with the west side or to put a gate into it. The latter arrangement was preferred, and the water now admitted to the slough under control is utilized in flooding considerable areas of grazing lands. The gate has a clear opening of 24 feet, divided by means of gate and bridge posts into four spaces, each of which can be closed by means of light drop timbers. The top of the gate is provided with a bridge floor, affording a crossing for the river road.

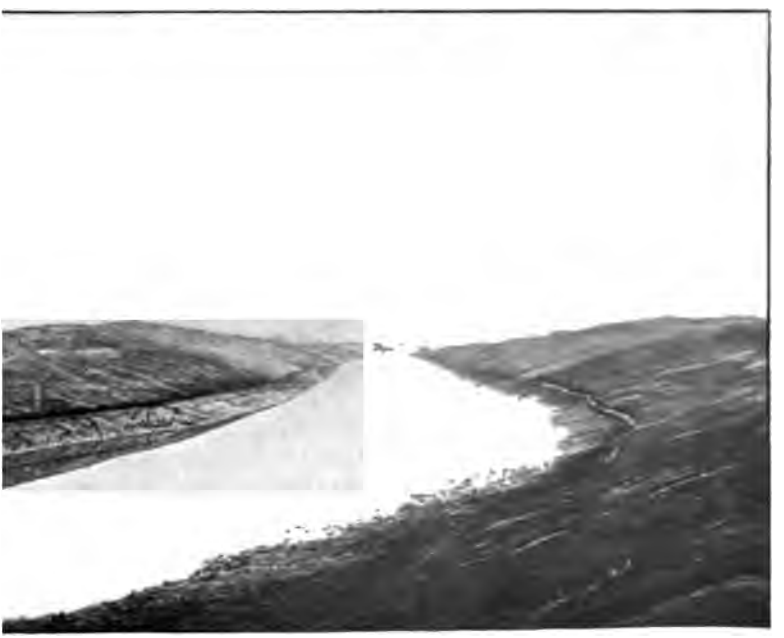
The slough continues inland for some distance as a well-defined channel through which water is led into a series of broad swales or gentle depressions. Across these depressions embankments of earth have been erected to a maximum height of 4 or 5 feet, by means of which the water is checked and made to inundate considerable areas. One embankment after another is cut for the passage of the water to the next, or the water breaks a passage of its own, and by this crude system of irrigation considerable areas of grazing land are wet during the spring rise of San Joaquin River.

It is hardly necessary to say that this method of wetting lands can be practiced only on large holdings, such as those of the Miller & Lux Company.

Temple Slough.—This is a slough of the same type as Posey Slough, though somewhat smaller. It has been similarly treated. It breaks out of the west side of the river about 3 miles below Posey Slough. Its head gate is also in use as a bridge, but the head gate is of the culvert type and is loaded on top with an earth fill. It is 16 feet wide, being divided by posts into four spaces. The bottom of the gate is at or slightly above the low-water plane of the river. The water admitted to the slough is used for irrigation in the same crude way as that of Posey Slough.

Sanjon de Santa Rita.—This is a third slough which breaks out through the west river bank, but, unlike Posey and Temple sloughs, it has a comparatively direct northwesterly course to a reunion with the main channel of San Joaquin River, about 12 miles below its head. There is no gate at its head for the control of inflowing water, but by means of temporary spur dikes or dams of sand and brush built into the main stream its period of flow is prolonged somewhat beyond what it would otherwise be. Water is diverted from the slough into a short ditch for the irrigation of several hundred acres of land at the Miller & Lux home place, the Santa Rita Ranch. The head of this ditch is about 3½ miles below the head of the slough, on its west bank. The flow of the water into the ditch, which has a width of 10 to 12 feet, is controlled by means of a light gate.

A.



B.

BLYTH CANAL.

A, Head at San Joaquin River; *B*, Nine-foot cut.

lyth Canal.—The main supply of water for the grassland irrigation ;he Chowchilla Ranch—already referred to in describing the Chowla Canal—is obtained from the San Joaquin River through the th Canal. (See Pl. III.) This is an east-side canal from San Joan River, the head of which is some miles below Las Juntas. The al is cut boldly into the river bank, with a bottom width of 20 feet. receives water from the river only at high stages. Its flow is conlled by a well-constructed regulator or head gate of the culvert type. supports a 5-foot fill of earth, which serves as a roadway across the ıal. The entrance to the culvert is divided into five 4-foot spaces by tical posts, which afford support to vertically sliding flashboards, 3 12 inches, raised by means of two double hooks engaging on iron ıs, to which power is applied by a winch. The canal discharges into east-side flat valley trough, where its flow is intercepted by a series cross levees, each extending from comparatively high ground at the er eastward across the depression. These cross levees, five or six in , are about a mile apart and are all equipped with stop gates, for discharge of surplus water to lower levels. The basin above the permost one is first filled, then water is allowed to flow through its p gate to the second, and so on to the last, thus effecting wholesale d very economical grass-land irrigation.

East Side or Stevenson-Mitchell Canal.—This canal, as its name implies, an east-side diversion. The head of the canal is at a point on San aquin River about 14 miles southwest of Merced. It has a northsterly course from this point for a distance of 20 miles, to near Merced ver, and commands a strip of land intermediate between the canal d San Joaquin River, 2 to 4 miles in width. (See Pl. IV.) The first w n iles of its course are over heavy clayey soils within the limits of e river's flood plain; the rest of its course is out upon the sandy ain which skirts the southern bank of Merced River. It commands out 50,000 acres of land.

The canal was constructed in 1887 by Messrs. J. J. Stevenson and e late John Mitchell, who were the principal owners of the lands comanded. It is 42 feet wide on the bottom and has side slopes of 1 to 1 the natural surface of the ground, generally at 4 feet above the botm. The fall is 6 inches to the mile. The canal levees rise about 3 et above natural ground and are set back 10 feet from the edge of the cavation. The proposed maximum water depth is 6 feet and width ı water surface 74 feet. The steep sides of the canal excavation were lopted as a result of the experience on the San Joaquin and Kings iver Canal—to prevent the growth of weeds and aquatic plants, which ke root readily on flat slopes.

All structures on the canal are of a simple flashboard type. There is rovided a regulator or head gate, and the flow of water in the canal is rther controlled by means of six check weirs, locally called stop ates. The canal *crosses the natural* drainage of the country. At *ch water course thus crossed* an inlet gate is put into the upper or

east-side canal levee and an outlet gate into the lower levee. These gates, 36 in all, range in width from 12 to 64 feet. All of them, as well as the check weirs in the canal, are bridged over. During winter the canal is out of service, and all of the gates are left open, to admit of the free passage of drainage waters across the line of the canal. The first cost of the canal work was about $80,000, which has been increased by subsequent expenditures, principally on the construction of branches, to about $125,000.

Water is sold from this canal by the acre irrigated, the charge being $2 per acre if application be made for water before the 1st of January, and $2.50 per acre if applied for later. This difference in the rate per acre is intended to induce irrigators to make application before it is known whether rainfall alone will prove sufficient to produce a grain crop.

The water distributed is under control of a canal superintendent. The canal company furnishes the gates for water delivery to irrigators. All lateral or branch canals are constructed by the irrigators and at their expense.

FRESNO RIVER.

HYDROGRAPHY.

Fresno River is a small torrential stream which drains the western slope of outlying spurs of the Sierra Nevada and which debouches upon the plains of San Joaquin Valley about 15 miles to the north of San Joaquin River. The river is dry in summer, but ordinarily has a considerable winter and spring flow. Its drainage basin above the point where it leaves the hills has an area of 272 square miles. Its flow usually commences in December and gradually increases till March. It ceases to flow in June, July, or August. During the rainy season it is subject to sudden rises of short duration. The maximum monthly average for the period 1878 to 1884, as approximated by the State engineer, is given as 568 second-feet for the month of March.

This stream soon after leaving the foothills proper bifurcates, sending off several lesser southerly branches, among which are Adobe and Cottonwood sloughs. A branch of the former reunites with the main stream some miles above Madera.

The bed of the river is a broad sandy depression, having comparatively high banks, near the eastern edge of San Joaquin Valley, where it is here and there flanked by small tracts of bottom land. At Madera the river banks are quite low, and a few miles farther west the freshet waters of the stream spread and are carried on toward San Joaquin River in several small channels.

CANALS AND DITCHES.

The water brought down to Madera in the flume of the Madera Lumber Company is used for irrigation at and to the west of Madera. This water is in large part a diversion from a tributary of San Joaquin River. It was necessary to make this diversion high up in the mountains and to carry the water from the one watershed across the dividing ridge into the other stream, on account of the small summer flow of Fresno River.

The lumber flume is of the ordinary V-shaped type, serving to float lumber a distance of about 35 miles, from the sawmill to Madera. The waste water from the flume flows into branches of the Madera Canal.

Madera Canal (*Fresno River Dam and Canal Company's Canal*).—This is a work of some magnitude which is interesting, in part, at least, on

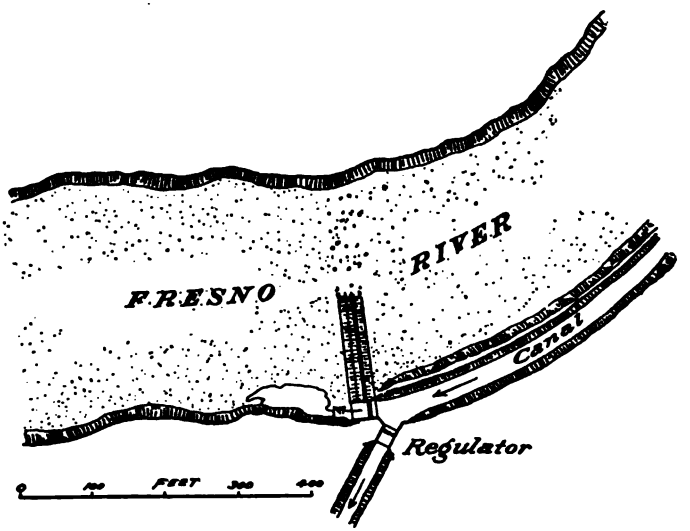


FIG. 7.—Head of Fresno River Canal.

account of the failures in which the first effort to secure water resulted. At great expense a dam was constructed across Fresno River about 2 miles above Madera, and from it a deep cut was excavated through the river bank, which there rises about 15 feet above the river bed. The dam consisted in the main of three rows of double sheet piling, reenforced by numerous ordinary piles, and a timber top work, forming a substantial sill across the stream that was intended to raise water 4 or 5 feet above the ordinary low-water plane. It was 311 feet long, and raised the water surface of the river 6 feet. It was intended also to intercept a subsurface flow supposed to exist in the sands of the river bed, but failed to develop any water from this source. A head gate and waste gate at the south end of this structure afforded control of the inflow into the canal, which was made 20 feet wide on the bottom,

with side slopes of 1 on 2. The depth of the cut through the river bank became less as distance from the river increased, and within a mile the canal was at the general surface level of the valley plain. From this point special effort was made to hold the main canal on the top of flat sand ridges, which position it was thought would afford the best possible command of the lands to be irrigated. Not only was the canal located on the highest available ground, but it was endeavored to hold the water surface in the canal and distributing ditches well above the natural surface of the ground. Material for canal banks was taken largely from the outside. After the expenditure of about $60,000 on the dam and canal it was found that breaks in the canal banks were of so frequent occurrence, and loss of water was so great, that it would be unwise to perfect or further extend the system. It was found, too, that the dam was a failure, particularly owing to the loss of water in the porous creek channel for miles above it. It was then found that by turning a portion of the river's flow into Adobe Slough, some miles farther upstream, and putting an inexpensive regulator in its head, water could be brought into the head of the canal and thence into a water course passing directly through the lands already prepared to receive it. This was done, and a few dams in the slough lower down, with regulating gates in each, completed a main canal system that has been comparatively inexpensive and has served its purpose well. About $20,000 was the first cost of this supplemental work.

It is understood that the water supply of the canal is further reenforced by the Adobe Ranch Ditch, which diverts water from the river at a point known as "Chinee Store," located in the foothills about 19 miles above Madera. This ditch is 10 feet wide on the bottom, carries water 4 feet deep, and at the Adobe Ranch, about 7 miles above Madera is connected with, and adds in a measure to the supply of, the lower works. The canal company proposes soon to construct storage works in the mountains in order to secure a continuous flow of water for its canal system.

The lands prepared for irrigation with water from this canal have an area of about 16,000 acres. They lie in a compact body to the southward, beyond Cottonwood Slough. The surface of the lands irrigated is very smooth and has an almost imperceptible slope of 7 to 8 feet per mile toward the southwest, adapting them admirably for irrigation. The soil is a sandy loam, rather heavy in the eastern portion of the irrigated region, and very sandy near its western limit. The heavier soils are underlain by a shaly hardpan a few feet below the surface. This is deeper below the surface of the sands or is entirely wanting.

Water is ordinarily available for irrigation from some time in December to May. Occasionally there is a good flow in the canal during June. It is diverted from the slough above referred to into branch ditches, which leave the slough just above the dams that have been built into it.

The method of irrigation is that of flooding in contour checks. The check levees are almost invariably constructed on contour lines, each being 1 foot lower than the next one above. The space between levees is divided by cross levees into compartments or checks whose area varies between wide limits, reaching as high as 40 acres. The ordinary check has an area of about 4 to 12 acres.

A good head of water for irrigation in this section is from 20 to 25 second-feet, but irrigators are frequently compelled to be content with only 10 to 15 second-feet. Irrigation is always commenced on the highest ground; the upper check is filled, and when full, the water is turned into the next lower check, while at the same time the surplus water of the upper check is also turned into the second check through a cut or gate in the contour levee between the two checks. Irrigation progresses night and day, and two men are constantly on duty for each irrigating head of water.

Both alfalfa and grain are generally irrigated but once each season. Grain as a rule is irrigated only when long-continued drought makes irrigation necessary to save the crop. Alfalfa receives more than one irrigation when water is available long enough.

The main canal of this system was until February of this year (1897) owned by a company or association of farmers which made no agreements for the delivery of water to irrigators for longer periods than one season. The distribution of water from the canal was in charge of a superintendent, to whom application for water had to be made before the irrigating season commenced. To these parties, who were required to specify the acreage which they purposed to irrigate and the time when water was required, the water was apportioned. When there was more water in the canal than was required by them, this was available for those parties who neglected to put in application for water before the date fixed as the commencement of the irrigating season. All parties applying for water were required to pay for it whether they used it or not. If water was insufficient to meet the demands of all applicants, it was apportioned by the superintendent according to his judgment, and was paid for by the acre of land actually irrigated. The charge for water was originally $1 per acre for grain and $2 for alfalfa. Subsequently the rate was raised to $1.50 for grain, and since 1885 it has been made $2, which is the charge per acre regardless of the nature of the crop. This rate remained the same no matter how often water was applied.

Of the 16,000 acres of land prepared for irrigation, it is reported that as much as 12,000 acres have been irrigated in one season. Ordinarily, however, the area irrigated is 5,000 to 6,000 acres.

The cost of land preparation for irrigation, covering the construction of small irrigating ditches and check levees, is estimated at $2 per acre, and the cost of each application of water at about 25 cents per acre.

The canal property has recently passed into the hands of a new corporation and will be managed on lines similar to those of the Fresno Canal from Kings River, except that in addition to selling water to lands covered by water rights water will also be furnished at an increased annual rate to lands not so covered. The new company is known as the "Madera Canal and Irrigating Company." It is capitalized at $400,000, in shares of $10 each.

The water rights are at present sold at $5 per acre. The annual water rate to be paid for each acre covered by the right is $1. The canal company places the gate in the canal bank for the delivery of the water; it reserves the right to convert the irrigator's private ditch into a branch ditch under the company's control, provided such use shall not interfere with the delivery of water to the owner of the water right; it makes the water right an appurtenance of the land on which it is located; it reserves the right to shut off water for repairs, and does not hold itself responsible for deficiency of water caused by drought, insufficient water in river, hostile diversion or obstruction, forcible entry, or temporary damage by flood or other accidents, but agrees to use at all times due diligence in restoring and protecting the flow of water in the canals. The canal company also reserves the right to sell 250 water rights of 1 second-foot of water each (1 second-foot to each 160 acres), and it is to be understood that each water right sold shall, whenever the canals flow less than 250 second-feet, represent the two hundred and fiftieth part of the available supply. The purchaser of the water right makes the water rights a lien upon the land covered; he agrees not to use water wastefully, and he grants to the canal company the right of way for canals and ditches over all land owned by him and located in the same township as the specific tract covered by the so-called water right.

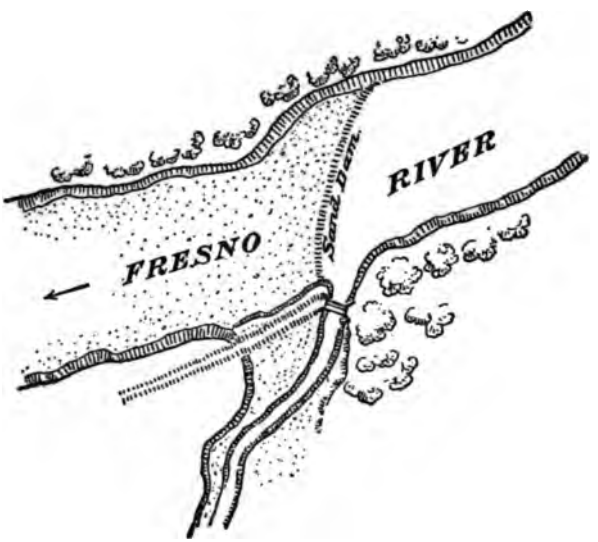


FIG. 8.—Head of Adobe Slough.

Water is delivered for $2 per acre per year to parties whose lands are not covered with water rights. The charge for grain land is $1 per acre for one wetting. The charge for garden irrigation is $5 per acre *per year.*

Adobe Ranch Ditch.—About 12 miles above Madera, on Fresno River, between low outlying foothills, lies the Adobe Ranch, on which between 300 and 400 acres of bottom lands are cultivated in alfalfa. Water for use on this land was made available by means of a small ditch constructed about fifteen years ago, the head of which is about 7 miles farther upstream, on the south side of the river. The water of this ditch was originally delivered into a small reservoir lying between low hills on the south side of Fresno River close by the tract to be irrigated, and water was drawn from the reservoir as required. The method of irrigation is by flooding in checks, each several acres in area. The cost of applying water is given at 65 cents per acre per irrigation.

The ditch is now said to be a feeder of the Madera Canal and is reported to have a bed width of 10 feet and to carry water 4 feet deep.

CHOWCHILLA RIVER.

HYDROGRAPHY.

The same description given for Fresno River will answer for the Chowchilla, except that its bank lands are even lower and the separation of its water into several channels soon after its debouchment on the valley plain is still more pronounced. The river's drainage basin is 268 square miles. Its flow and period of flow are about the same as noted for Fresno River.

Two channels break out from the south side of the river near the eastern margin of the valley plain. These are Berenda and Ash sloughs, both of which have a very direct southwesterly course, being respectively 4 and 2 miles from the main stream at Minturn. All of these channels across the east-side plain of San Joaquin Valley have shallow beds of sand and banks rather low, but still somewhat higher than the general surface of the plain. Their flow spreads over large areas near San Joaquin River, and there is no pronounced junction of Chowchilla River with the main drainage way of the valley.

CANALS.

Kohler, West & Minturn canal system.—The use of water from Chowchilla River for irrigation has been confined to several thousand acres of land at and near Minturn. (See map, Pl. IV.) The main river channel has been converted into a supply canal by the construction of several dams. The uppermost of these dams is just below the head of Ash Slough. It is in the nature of an earth or sand embankment carried across the river channel and equipped with a large regulating gate in its center. The gate is of the ordinary flashboard type, with a light bridge over its top. It is protected against undermining by sheet piling, and the water required for irrigation below is allowed to pass through it. Any excess flow of the river is turned down Ash and Berenda sloughs. About a mile north of Minturn and about 10 miles below the dam just referred to, a second dam of similar character is

maintained. This structure converts a section of the river into a reservoir which overflows through a gate in the dam to the lower portion of the river, and through gates in river-bank levees sends water toward the north and south.

The water diverted at this point is used principally on the vineyard lands of Messrs. Kohler, West & Minturn. Some of it is passed on to other irrigators. The irrigation ditches are small, 8 to 12 feet wide, carrying water about 2 feet deep, and terminate within a few miles of the river.

A third dam is maintained in the river about 2 miles below the second, and water is taken from above it for the irrigation of the Minturn farm.

These dams are 8 to 14 feet high, built of such material as the river bed affords, and have stood remarkably well. The success of this system of handling the water of the river, which is 200 to 300 feet

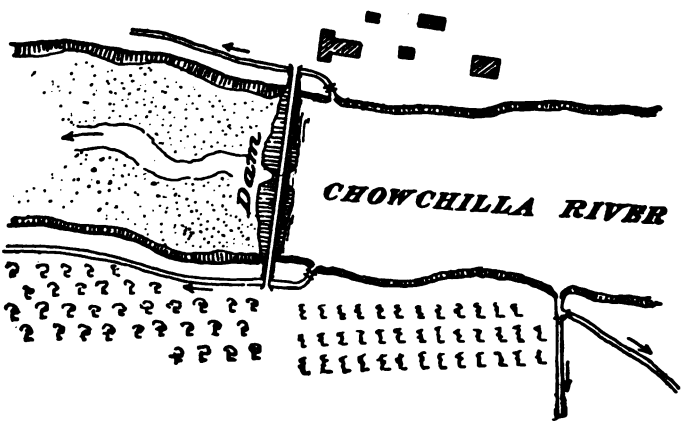


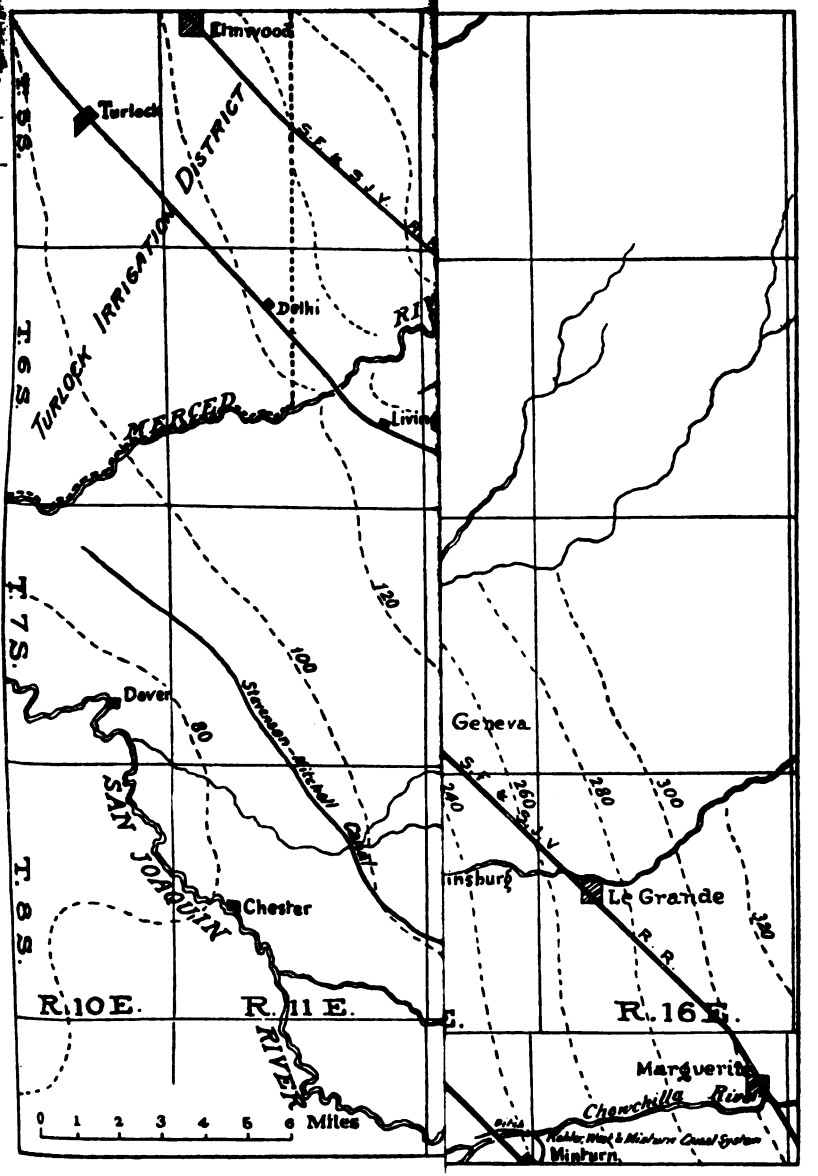
FIG. 9.—Water diversion by Kohler, West & Minturn Canal.

wide, depends, of course, upon the possibility of forcing practically the entire freshet flow into the other channels.

The method of vineyard irrigation practiced at Minturn is that of flooding in small checks, some of which have been laid out rectilinearly, while others are fitted to the contours of the ground. The soils irrigated are sandy loams, very retentive of moisture, which with water have proved very productive and well adapted to the culture of the grape.

Surplus waters—that is, water in excess of that required for the irrigation of the lands belonging to the firm owning the canal—have been held for sale. The rate which prevailed several years ago was $1.50 per acre of land actually irrigated.

Buchanan Reservoir project.—A work of considerable magnitude proposed for Chowchilla River is the construction of a reservoir of great capacity several miles above Buchanan. This work is to be carried out to secure an adequate water supply for the 30,000 acres of land of


Elmwood
Turlock
TURLOCK IRRIGATION DISTRICT
S.F. & S.J.V. R.R.
Delhi
MERCED
Livin
T. 5 S.
T. 6 S.
T. 7 S.
T. 8 S.
120
100
80
Dover
Stevenson-Mitchell Canal
SAN JOAQUIN
Chester
RIVER
R. 10 E.
R. 11 E.
R. 16 E.
0 1 2 3 4 5 6 Miles
Geneva
240
260
280
300
320
Le Grande
R. R.
Marguerite
Chowchilla River
Kohler, West & Minturn Canal System
Minturn

maintained. This structure converts a section of the river into a reservoir which overflows through a gate in the dam to the lower portion of the river, and through gates in river-bank levees sends water toward the north and south.

The water diverted at this point is used principally on the vineyard lands of Messrs. Kohler, West & Minturn. Some of it is passed on to other irrigators. The irrigation ditches are small, 8 to 12 feet wide, carrying water about 2 feet deep, and terminate within a few miles of the river.

A third dam is maintained in the river about 2 miles below the second, and water is taken from above it for the irrigation of the Minturn farm.

These dams are 8 to 14 feet high, built of such material as the river bed affords, and have stood remarkably well. The success of this system of handling the water of the river, which is 200 to 300 feet

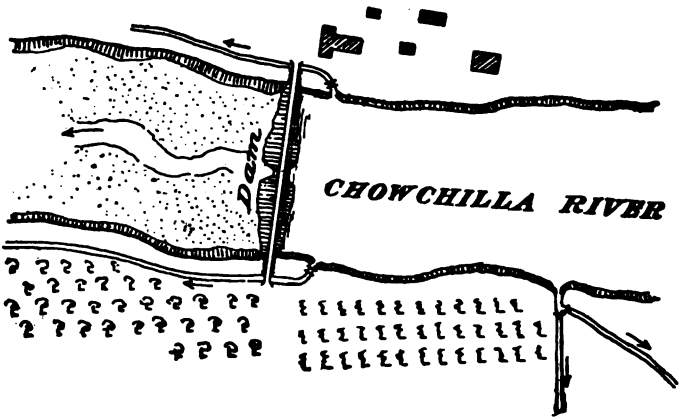


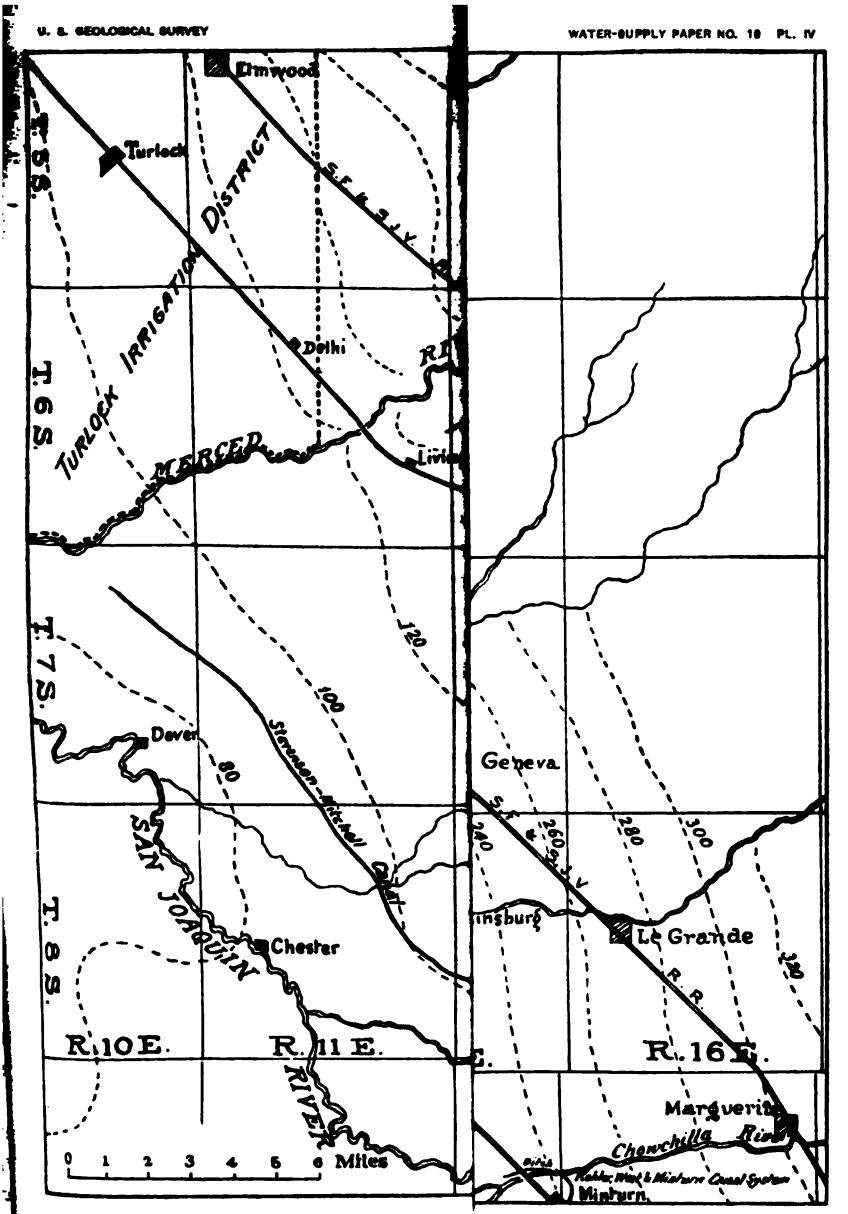
FIG. 9.—Water diversion by Kohler, West & Minturn Canal.

wide, depends, of course, upon the possibility of forcing practically the entire freshet flow into the other channels.

The method of vineyard irrigation practiced at Minturn is that of flooding in small checks, some of which have been laid out rectilinearly, while others are fitted to the contours of the ground. The soils irrigated are sandy loams, very retentive of moisture, which with water have proved very productive and well adapted to the culture of the grape.

Surplus waters—that is, water in excess of that required for the irrigation of the lands belonging to the firm owning the canal—have been held for sale. The rate which prevailed several years ago was $1.50 per acre of land actually irrigated.

Buchanan Reservoir project.—A work of considerable magnitude proposed for Chowchilla River is the construction of a reservoir of great capacity several miles above Buchanan. This work is to be carried out to secure an adequate water supply for the 30,000 acres of land of


Elmwood
Turlock
TURLOCK IRRIGATION DISTRICT
S.F. & S.J.V.
Delhi
MERCED
Livi
T. 5 S.
T. 6 S.
T. 7 S.
T. 8 S.
120
100
80
Dover
Stevenson-Mitchell Canal
SAN JOAQUIN
Chester
RIVER
R. 10 E.
R. 11 E.
R. 16 E.
Geneva
S.F. & S.J.V.
240
260
280
300
320
Le Grande
R.R.
Marguerite
Chowchilla River
Kahler, West & Minturn Canal System
Minturn
0 1 2 3 4 5 6 Miles

the Berenda-Chowchilla tract owned by the Sharon estate. The dam as proposed is to be built of uncoursed rubble. It is to have a height of 100 feet and will store 42,400 acre-feet of water.[1]

MERCED RIVER.

HYDROGRAPHY.

Merced River is to be classed among the principal streams which flow down the western slope of the Sierra Nevada into San Joaquin Valley. It has a perennial flow, its low-water discharge being about the same as that of Tuolumne, Stanislaus, and Mokelumne rivers, nearly 200 second-feet. The river has a high mountain and foothill watershed of 1,076 square miles. Its bed where it enters the valley is considerably depressed below the general surface of the plain. It is flanked by low bottom lands, which extend with irregular width to the base of the bluffs rising to the higher land of the main valley. The bottom lands near Snelling expand to a width of several miles, forming a secondary valley of no small importance. Westward from Livingston the main plain gradually drops to the level of the river lands, and long before a junction with San Joaquin River is effected the Merced is flanked by large areas subject to periodical inundation.

The flow of the river approximated by the State engineer for the years 1878 to 1884 indicates an average monthly rate of 1,500 second-feet for February, over 1,700 second-feet for March, 3,000 to 5,000 second-feet from April to June, 1,900 second-feet in July, 400 to 600 second-feet in August, December, and January, and a low-water flow of about 200 second-feet during the months of September, October, and November.[2]

These figures, indicating as they do a very considerable flow in the spring and early summer, show the importance of this stream as a possible source of water supply for large irrigation canals. Although the value of this stream for irrigation has long been recognized, as well as that of the other rivers of the same class—the Tuolumne, Stanislaus, and Mokelumne—the construction of irrigation works on a large scale has progressed but slowly. This is due, first, to the fact that the position of these streams, depressed as they are from 40 to 80 feet below the general surface of the east-side plain, makes a diversion of water upon the higher surface of this plain very expensive; and, secondly, to the fact that the lands commanded are sufficiently favored with natural advantages to render them available, in a measure at least, for farming without rrigation.

The only canal from Merced River which carries water out upon the east-side plain of San Joaquin Valley is the Merced or Crocker-Huffman Canal. All other irrigation works are comparatively small ditches in use for the irrigation of the limited bottom lands.

---

[1] See American irrigation engineering, by H. M. Wilson: Thirteenth Ann. Rept. U. S. Geol. Survey, Part III, 1893, p. 314.

[2] Physical Data and Statistics, W. H. Hall, State engineer of California, 1886, pp. 434, 476.

CANALS AND DITCHES.

Merced or Crocker-Huffman Canal.—This work is the extension and enlargement of an older ditch known as the Farmers Canal. It has its head on the south side of Merced River, about 2 miles below Merced Falls. (See map, Pl. IV.) Skirting the base of the bluffs to the south of the bottom lands, it reaches the upland and, penetrating several foothill spurs, attains a position which commands the east-side plain of San Joaquin Valley to and beyond Bear Creek. (See Pl. VIII.)

The enterprise was conceived by the late William G. Collier, a surveyor and civil engineer, who had already had some experience in constructing irrigation ditches from Bear Creek. An organization of farmers was formed, which incorporated in 1873 under the name of the Farmers Canal Company, and work on the canal was pushed forward as rapidly as the available means would permit. Unforeseen difficulties, notably due to lack of funds occasioned by excessive cost of excavation, prevented the plans from being fully carried out as originally proposed. However, by March, 1876, the canal reached Canal Creek, and water was, through it, made available for irrigation. The length of the canal to this point was about 8 miles. For most of the way it had been cut into a hard gravelly soil, which cost to excavate about 20 cents per cubic yard, instead of 10 cents, as had been estimated. It penetrated a ridge by a tunnel 1,600 feet long, 11½ feet wide by 9 feet high, which cost about $20,000. This tunnel is in soft sandstone and firm cemented gravel, and did not require lining. The canal as constructed had a bed width of 20 feet, and was 4 feet deep. It followed, in a southwesterly direction, a grade line having a fall of 1 foot to the mile, except at the tunnel approaches and in the tunnel itself, where the fall was increased to 10 feet to the mile.

The Farmers Canal Company could not succeed in making arrangements for the extension of the canal beyond Canal Creek, and finally sold out to Messrs. Charles Crocker and C. H. Huffman in 1882. The enlargement of the canal was commenced at once. Except one change of location near the head of the canal, the original alignment of the old ditch was all too carefully adhered to. In making the enlargement to a bed width of 60 feet, the material handled, including original levees and spoil banks, was fully as great as it would have been on a new route adapted to the increased canal dimensions, and it is questionable whether a complete change of alignment for some of the canal sections would not have been advisable. The tunnel, too, was enlarged to a width of 22 feet, while it might well have served for many years without enlargement.

Near its head the new canal was made 80 feet wide on the bottom. It was planned to have a depth of 10 feet throughout. Its extension beyond Canal Creek was rapidly pushed. The creek channel was enlarged, and for about a mile became a section of the canal. From

REGULATOR IN CROCKER-HUFFMAN CANAL.

ıat point the main canal was given a more southerly course, skirt-ıg the western base of a number of foothill spurs and finally pene-:ating one at a point about 8 miles north of Merced, in a timber-lined ınnel 20 feet wide, 13 feet high, and 2,100 feet long, with a total fall f about 5 feet. Tunnel frames are constructed of timbers, 12 by 14 ıches, 4 feet apart. Lagging and flooring are 3 inches thick, all edwood. Tunnel lining required over 1,000,000 feet of timber, and it lone is said to have cost about $30,000. It is extremely doubtful 'hether the reduction of a few miles in the length of the canal could ıstify its construction, even if it had been found to require no lining. 'he total cost of the tunnel was about $70,000. Work on it progressed ight and day, and it was completed in about six months. (See Pl. VII.)

About 6 miles below the tunnel just described, and about 12 miles y canal from Canal Creek, the canal delivers its water into a reser-oir, the primary purpose of which is to afford a reliable supply of rater to the city of Merced, lying about 6 miles farther to the south-rest. This reservoir has a surface area of about 600 acres. It lies etween two low foothill spurs, the gap between which was closed by ıeans of an earth dam having a crest length of 2,200 feet and a maxi-ıum height of 50 feet. The dam rests on a very firm foundation of emented gravel, into which a wide, deep puddle trench was cut. Much f the material of which it is constructed had to be loosened by blast-ıg. All material was wet when put into place, and was firmly tram-led by horses. There was enough clay in the material used throughout he structure to make it a very compact bank.

A masonry conduit, brick laid in cement mortar, has been carried ırough the natural foundation under the dam, and in this the delivery ipe and the sluice pipe have been placed. (See Pl. IX.) The main canal continued southward, or rather southeastward, from the reservoir on continuation of the grade line entering the reservoir from the north. his arrangement cuts the canal extension out of a supply when the servoir is not full. Water was turned into the reservoir through the ompleted main canal on February 1, 1888.

The diversion from the river is made at a point where Merced River aves the base of a vertical cliff which rises to the south. The canal ead gate or regulator is constructed between two rock-filled timber iers or bulkheads, of which the southernmost rests against the foot this cliff. The regulator has a width of 80 feet, divided into a num-er of small spaces between vertical posts, all of which are controlled y means of vertically sliding gates. (See Pl. VI.) From this gate a mber dam, raising the water surface about 8 feet, extends, with a road downstream sweep, across the river to a connection with the bot-om land on the north side of the river. (See Pl. V.) The dam proper ests on a timber platform or sill, which, extending downstream 10 or 12 eet below the lower face of the dam, forms an apron, on which the over-alling water is received. Considerable trouble has been experienced in pairing cuts which the river has made through the bottom lands around

Berenda-Chowchilla tract owned by the Sharon estate. The dam proposed is to be built of uncoursed rubble. It is to have a height 00 feet and will store 42,400 acre-feet of water.[1]

MERCED RIVER.

HYDROGRAPHY.

Merced River is to be classed among the principal streams which flow rn the western slope of the Sierra Nevada into San Joaquin Valley. has a perennial flow, its low-water discharge being about the same that of Tuolumne, Stanislaus, and Mokelumne rivers, nearly 200 ond-feet. The river has a high mountain and foothill watershed of 76 square miles. Its bed where it enters the valley is considerably pressed below the general surface of the plain. It is flanked by low tom lands, which extend with irregular width to the base of the ffs rising to the higher land of the main valley. The bottom lands ar Snelling expand to a width of several miles, forming a secondary lley of no small importance. Westward from Livingston the main in gradually drops to the level of the river lands, and long before a ction with San Joaquin River is effected the Merced is flanked by ge areas subject to periodical inundation.

The flow of the river approximated by the State engineer for the ars 1878 to 1884 indicates an average monthly rate of 1,500 second-t for February, over 1,700 second-feet for March, 3,000 to 5,000 ond-feet from April to June, 1,900 second-feet in July, 400 to 600 ond-feet in August, December, and January, and a low-water flow of out 200 second-feet during the months of September, October, and vember.[2]

These figures, indicating as they do a very considerable flow in the ing and early summer, show the importance of this stream as a pos-le source of water supply for large irrigation canals. Although the ue of this stream for irrigation has long been recognized, as well as t of the other rivers of the same class—the Tuolumne, Stanislaus, Mokelumne—the construction of irrigation works on a large scale progressed but slowly. This is due, first, to the fact that the posi- of these streams, depressed as they are from 40 to 80 feet below general surface of the east-side plain, makes a diversion of water n the higher surface of this plain very expensive; and, secondly, to fact that th commanded are sufficiently favored with natu- advantage r them able, in a measure at least, for ning wit

he only erced ich carries water out upon the -side quir he Merced or Crocker-Huffman al. tion comparatively small ditches in the tom lands.

n: Thirteenth Ann. Rept. U. S. Geol. Survey,

neer of California, 1886, pp. 434, 476.

6 miles north of Merced. These lands were sold to a colony of Hollanders, who failed to achieve the results anticipated. They experienced great difficulty in preparing and cultivating the shallow, coarse, gravelly, hogwallow soil, which, unlike the black soil of the Bear Creek country nearer Merced, or the sandy loams of Atwater and Livingston, had before canal construction been considered worthless, except for grazing purposes. The efforts of the colonists proved unavailing, and after a time it was realized that the cultivation of the selected tract in small areas of fruits, truck gardens, and the like, could not be made profitable, and the colony project was abandoned.

This unsatisfactory experience, together with the general business depression of the last few years, will account in large measure for the comparatively small demand that has been made for water from the Merced Canal. The area actually irrigated in a season has probably not exceeded 12,000 acres.

MERCED RIVER BOTTOM CANALS.

The bottom lands flanking Merced River, extending downstream from Merced Falls to near Livingston, are in large part irrigated by a number of small ditches, chiefly interesting from the fact that some of them are among the oldest irrigation works in San Joaquin Valley.

Merced Falls mills.—Power is supplied to a flour mill and to a woolen mill at Merced Falls by water from Merced River, which there crosses a ledge of slate. The natural fall of the rapids at that point is increased by means of a light dam, about 800 feet long, which follows the crest line of the slate reef from bank to bank. This dam is of the lightest possible type, being constructed of a series of inclined timbers, usually 4 by 6 inches, loosely resting upon the rock and supported from below by means of shores, irregularly spaced according to facility of support afforded by the uneven surface of the rock. The spaces between timbers are closed with 1-inch boards. The maximum height of the structure is about 8 feet. Power has been developed at the falls since about 1856.

Merced Falls Ditch.—The water from this ditch is taken from the north side of Merced River above the dam at Merced Falls, but the rights of this ditch are subject to the prior rights of the mills. The ditch was constructed in 1857 and has been in use to some extent ever since. It is about 2 miles long, 4½ feet wide on the bottom, 1½ feet deep, and has a fall of about 2 feet to the mile. The area irrigated by it is 300 to 400 acres. Like all the ditches in Merced Bottoms, it is a private ditch, supplying water only to lands of its owners.

Murray Mill Ditch.—This is another small north-side ditch, which has its head about 1½ miles below Merced Falls. A low dam of cobbles and brush is annually thrown across the river to force water into the ditch, which is about 3 miles long. The ditch has a width of about 10 feet, carrying water 1 to 1½ feet deep, and supplies water to about 100 acres

of land. It also furnishes power for a small gristmill located abou mile below its head. The ditch was constructed about 1858, in whi year rights of way for it were secured, but it fell into disuse and w reconstructed or enlarged in 1862. The dimensions originally intend and which were probably given the earlier ditch, were described as f lows: 5 feet wide on bottom, 7 feet on top, and deep enough to carr feet of water.

Snelling Ditch.—The Snelling Ditch was constructed in 1860, or ab that time, for the irrigation of lands near Snelling. A company w formed composed of interested landowners, and ownership in the di was represented by shares of stock. The head of the ditch is on t north side of the river, about 2 miles above Snelling, which was at t time of the ditch construction a prosperous town and the first cou seat of Merced County. A temporary dam of cobbles and brush annually constructed at the head of the ditch. The ditch is abou miles long, 6 feet wide on the bottom, and carries water from 1 to 2 f in depth on a grade of about 2 feet to the mile. The irrigation is co fined principally to orchards, vegetables, and alfalfa, Chinese being t renters of most of the land irrigated.

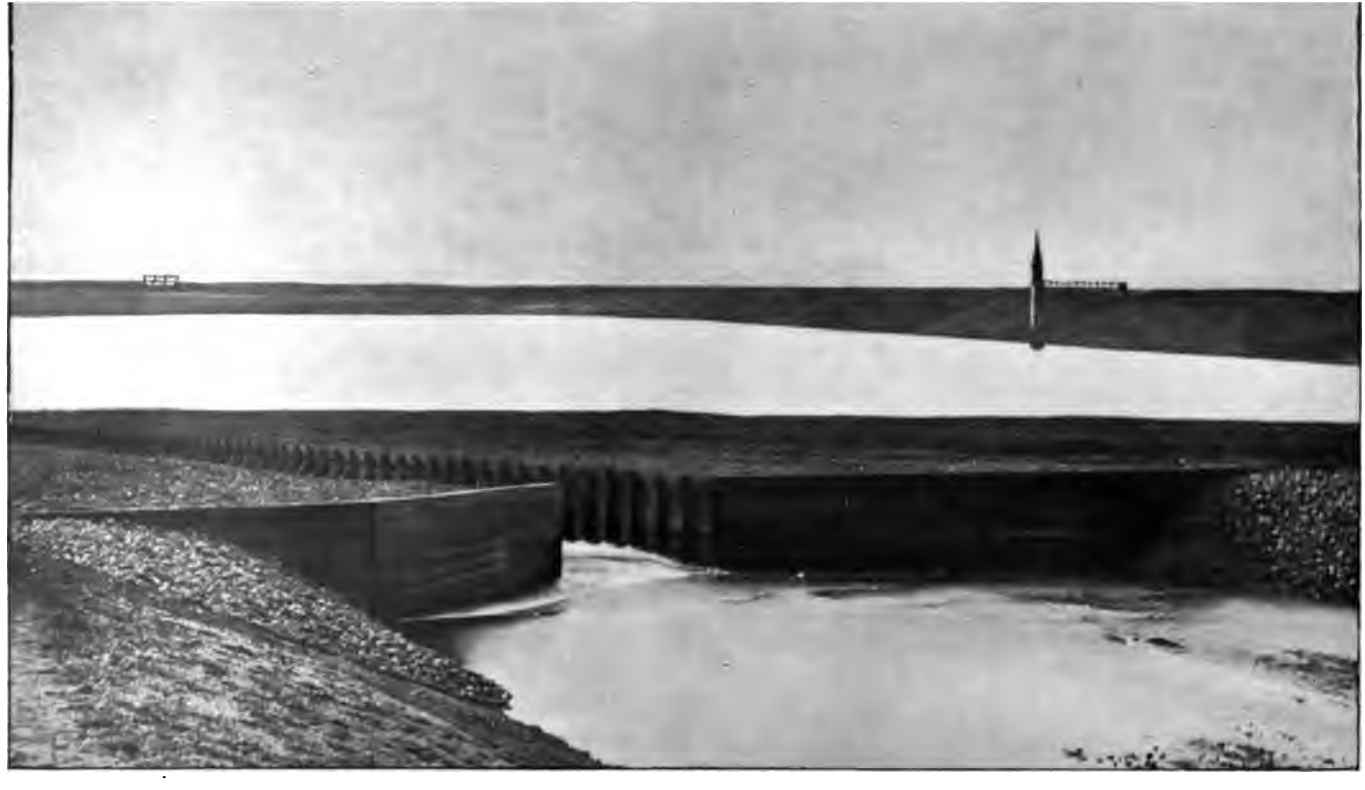
Montgomery & Anderson Ditch.—This is another old-time ditch small capacity. Its head is on the north side of Merced River, at t ford, about a mile above Snelling. It is about 1½ miles long, and su plies water to a few acres of alfalfa, orchard, and gardens near Snellin Its bed width is only about 4 feet, and the depth of water carried about 1 foot.

Ingalsbe Slough.—This is a natural high-water channel, which brea out through the north bank of Merced River just above Snelling. has a westerly course along or near the northern margin of the bott lands for about 8 miles, and supplies water to several small ditches, no bly the Ruddle ditches, besides wetting considerable areas by infilt tion of water from the slough into the porous subsoils. A barrier h been erected across the head of the slough, and in this a regulating ga is maintained.

Scott Ditch.—The head of this ditch is on the north side of Merc River, about a mile below Snelling. It was constructed in 1857, ha bottom width of about 3 feet, and is about 2 miles long.

Dean Ditch.—This is still another small ditch, on the north side of t river. It takes water about 1½ miles below Snelling, is about 3 mi long, 5 feet wide on the bottom, and 1 to 2 feet deep. Some of its wa is dropped into Hopeton Slough.

Old River.—This is an old channel about 3 miles below Snelli During the freshets of 1861–62 Merced River cut a new channel n the southern margin of its bottom lands, abandoning about 3 to 4 m of its old channel. This is now known as Old River, and a cut been made at its head to increase its flow and to make it serve a source of supply for the irrigation of a number of farms. A little o



INLET TO MERCED RESERVOIR, CROCKER-HUFFMAN CANAL.

miles below its head water is diverted from it into the Sillman & Ruddle Ditch, in which it is carried westerly about 3 miles. This ditch has a bed width of 7 feet, and carries water 1 to 2 feet deep. It has a fall of about 7 feet to the mile. Half a mile below the head of the Sillman & Ruddle Ditch is the Means or Montgomery Ditch, which irrigates a few acres of land lying between Old River and the present channel of Merced River.

Feldhaus & Reed Ditch.—This is a south-side ditch, having its head about 2 miles above Snelling, just below the head of the Snelling Ditch. It is between 2 and 3 miles long, and supplies water for the irrigation of several hundred acres of bottom land on the south side of the river, opposite Snelling. The ditch penetrates the point of a hill in a tunnel about 400 feet long, 3 feet wide, and 5½ feet high. The ditch was constructed in 1865, and was made 6 feet wide on the bottom, 1½ feet deep, and has a fall of nearly 5 feet to the mile. All parties interested in this ditch contributed toward the construction of that part of it located above the point where it leaves the land of each, in proportion to individual interests. The expense of maintenance is apportioned in the same way. No water is sold.

Hamlin or Millrace Ditch.—Water is diverted from the south side of Merced River into this ditch about a mile above Snelling. It has a course parallel with the lower portion of the Feldhaus & Reed Ditch, but extends beyond it, being about 2½ miles long. It is 7 feet wide on the bottom, carries nearly 2 feet of water, and has a fall of between 4 and 5 feet to the mile. The ditch was constructed for irrigation purposes. Its owners were subsequently interested in the erection of the Merced flour mill, and the ditch became a part of the mill property. Some concessions to the use of its water were made for right-of-way privileges. Irrigators use the waters of the ditch on Sundays, at night, and at other times when the mill is not running. Several hundred acres of alfalfa and orchard are dependent on it for a water supply.

Griffith & Shaver Ditch.—This is a ditch on the south side of Merced River, about 10 miles by river below Snelling. It is a small ditch, about 2½ miles long and 3 to 4 feet wide. It has been poorly maintained, and has been out of service at various times.

Other small ditches, besides those enumerated, have been in service in Merced Bottoms; but it is impossible to obtain full and accurate data relating to them or to many of those mentioned, for the reason that official records disclose nothing more, as a rule, than a claim to water antedating ditch construction or possibly a right-of-way grant, and all other facts relating to the history of irrigation must be compiled from the recollections of the oldest inhabitants.

METHODS OF WATER DIVERSION AND USE.

Except in the case of the Crocker-Huffman Canal and the Merced Falls Ditch, the diversion of water from Merced River is accomplished by means of *temporary dams constructed* of brush and cobbles. These

are, as a rule, renewed after the high waters of each season, though occasionally they serve through several years. The inflow of water into the ditches is generally regulated by means of primitive wooden gates. In some cases overfalls or escape ways near the heads of the ditches reduce the amount of water to ditch capacity.

Before the flood year of 1861–62 Merced River had a well-defined channel, with high banks, from Merced Falls to near Livingston. The high water of that year, however, washed away large tracts of the alluvial soil of the bottoms, leaving in their stead great cobble beds; it caused a general silting up and widening of the river channel and a consequent rise of the plane of the water surface, as well in the river as in the adjacent soils of the bottom lands. Since that time it has been an easy matter to accomplish a diversion of water for use on the bottom lands of the river.

On these lands the crops generally irrigated are fruits, grapes, hops, alfalfa, and vegetables. Many of the orchards are not well cared for, being leased to Chinese from year to year. Alfalfa is irrigated by flooding in small checks; orchards and vineyards in furrows. Irrigations are frequent. They commence with the cessation of rains and continue at intervals of three or four weeks for orchards, vineyards, etc., until their product is matured. Each new growth of alfalfa receives at least one wetting. Ground in which grain is to be planted is sometimes wet before plowing. To some extent lands are moistened by the rise of water in the porous subsoils, caused by infiltration from the irrigation ditches and water courses. The ditches are owned by the landowners, and the cost of water from each is a proportional part of the cost of maintaining the works. Water is generally used in turn by those irrigators dependent upon the same ditch for their water.

TUOLUMNE RIVER.

HYDROGRAPHY.

The description written for Merced River applies also to this stream. Its mountain drainage basin is, however, somewhat larger, covering 1,635 square miles, and the extent of its bottom lands is considerably less than those of Merced River.

Its low-water flow at the mouth of its canyon above Lagrange ordinarily covers the period August to December. The winter rise of the stream, due to rainfall in the foothills and low mountain spurs within its watershed area, is immediately followed by the spring rise, due to the melting of snow in the higher mountains. The flow of the stream gradually increases till May or June, and falls rapidly in the latter part of June and in July. The maximum average monthly flow, based upon the approximations of the State engineer department, for the six years 1878 to 1884, has been noted at 4,500 second-feet for May.[1]

[1] *Physical Data and Statistics*, W. H. Hall, State engineer of California, 1886, pp. 431, 476.

The extensive freshet flow, which occurs in winter, when warm rains fall upon snow, may reach 120,000 second-feet.

There has been but little use made of water from this stream for irrigation, but works on a large scale have been commenced which are quite in keeping with its importance as a source of supply.

Two gaging stations have been maintained on this river by the United States Geological Survey, one at Lagrange, near where the river issues from the foothills, and the other at Modesto, farther down the valley.[1] Gage readings at the lower station have been maintained by the Southern Pacific Company for a number of years, and an attempt was made to rate the station, but owing to its proximity to San Joaquin River, whose back flow in times of high water affected the height at Modesto, the work was not so satisfactory as it might be.

For this reason the other station at Lagrange was established August 30, 1895. Its location is at the highway bridge in the town. The following facts in regard to the discharge of the river are brought out:

At Modesto:

1895. Maximum discharge, February, 19,650 second-feet; minimum, October, 120 second-feet; average for the year, 3,719 second-feet.

1896. Maximum discharge, April, 14,873 second-feet; minimum, October, 180 second-feet; average for the year, 2,927 second-feet.

Estimated monthly discharge of Tuolumne River at Modesto, California.

[Drainage area, 1,501 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1895.						
January	16,130	1,398	4,828	296,900	3.700	3.220
February	19,650	1,322	3,915	217,430	2.720	2.610
March	10,785	1,710	3,165	194,610	2.430	2.110
April	10,012	3,255	5,824	346,570	4.330	3.880
May	16,000	6,097	11,798	725,400	9.065	7.870
June	12,530	5,616	9,163	545,110	6.815	6.110
July	6,692	1,550	3,831	235,540	2.944	2.554
August	1,550	285	848	52,125	0.652	0.565
September	5,616	200	615	36,600	0.457	0.410
October	200	120	152	9,346	0.117	0.101
November	500	210	255	14,985	0.187	0.170
December	730	180	283	17,405	0.217	0.189
The year	19,650	120	3,719	2,692,021	33.634	2.45
1896.						
January	13,686	246	3,080	189,370	2.37	2.05
February	1,880	950	1,182	68,007	0.86	0.79
March	11,798	1,006	2,725	167,578	2.10	1.82
April	14,873	1,970	3,577	212,828	2.66	2.38
May	14,754	2,550	5,180	318,500	3.98	3.45
June	14,754	6,990	11,648	693,104	7.87	7.09
July	8,735	1,306	4,121	253,408	3.17	2.75
August	1,118	300	575	35,392	0.44	0.38
September	2,918	220	574	34,191	0.43	0.38
October	474	180	224	13,797	0.17	0.15
November	7,802	273	1,210	72,012	0.90	0.81
December	3,560	516	1,028	63,247	0.79	0.69
The year	14,873	180	2,927	2,121,434	25.74	1.90

[1] Twelfth Ann. Rept. U. S. Geol. Survey, Part II, 1891, p. 322; Bull. No. 131, 1895, p. 83; Bul. No. 140, 1896, pp. 298-304; Eighteenth Ann. Rept., Part IV, 1897, pp. 378-385; Water-Supply and Irrigation Paper No. 11, 1897, p. 90; Water-Supply and Irrigation Paper No. 16, 1898, pp. 188-189.

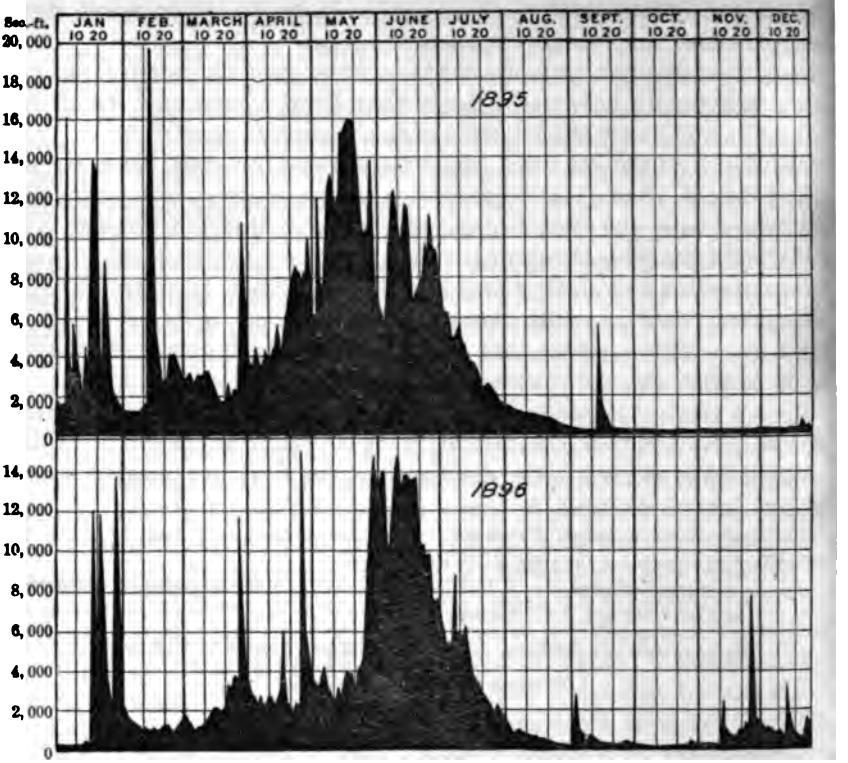


Fig. 10—Discharge of Tuolumne River at Modesto, California, 1895–96.

Estimated monthly discharge of Tuolumne River at Lagrange, California.

[Drainage area, 1,501 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1895.						
September	2,182	78	263	15,635	0.195	0.175
October	299	78	134	8,231	0.103	0.089
November	129	112	113	6,698	0.084	0.075
December	1,004	129	270	16,614	0.208	0.180
1896.						
January	7,610	350	2,312	142,159	1.78	1.54
February	1,780	1,080	1,164	66,954	0.84	0.78
March	11,798	1,006	2,725	167,578	2.10	1.82
April	7,990	2,670	3,522	209,574	2.60	2.34
May	10,100	2,670	4,429	272,371	3.40	2.95
June	9,320	5,140	7,692	457,705	5.74	5.13
July	5,330	960	3,003	184,677	2.31	2.00
August	3,050	227	485	29,828	0.37	0.32
September	1,620	132	432	25,742	0.32	0.29
October	350	75	19	7,366	0.09	0.08
November	5,710	227	1,135	67,543	0.83	0.75
December	3,620	670	1,083	66,603	0.83	0.72
The year	11,798	75	2,342	1,698,100	21.21	1.56

ted monthly discharge of Tuolumne River at Lagrange, California—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1897.						
.....	3,930	660	1,231	75,692	0.94	0.82
.....	13,200	1,840	5,172	287,238	3.59	3.45
.....	10,800	2,500	4,032	247,920	3.10	2.69
.....	13,500	3,280	7,735	460,263	5.75	5.15
.....	14,700	9,600	11,923	732,121	9.15	7.94
.....	9,300	2,500	5,673	337,566	4.22	3.78
.....	4,840	480	2,181	134,105	1.67	1.45
.....	570	76	237	14,573	0.18	0.16
r.....	140	28	86	5,117	0.07	0.06
.....	750	76	222	13,650	0.17	0.15
r.....	3,540	260	768	45,699	0.57	0.51
.....	4,060	570	1,104	67,883	0.85	0.74
year.....	14,700	28	3,364	2,422,827	30.26	2.24

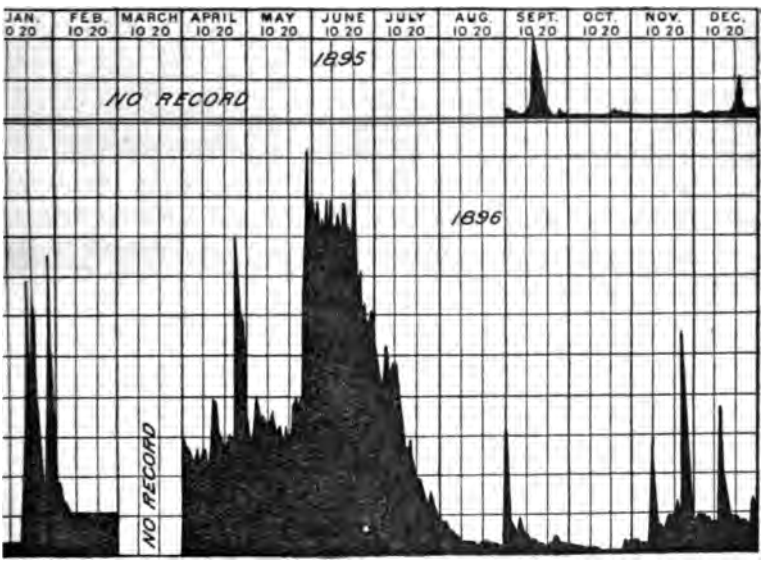


FIG. 11.—Discharge of Tuolumne River at Lagrange, California, 1895–96.

Lagrange:

Maximum discharge, March, 11,798 second-feet; minimum, October, 75 sec-t; average for the year, 2,342 second-feet.

Maximum discharge, May 25, 14,700 second-feet; minimum, September 21, 28 feet; average for the year, 3,364 second-feet.

CANALS AND DITCHES.

range Ditch.—In 1871 this ditch was constructed by the Lagrange and Hydraulic Mining Company, from the upper end of Indian French Hill, at Lagrange, for mining purposes, and has been in nt use, supplying water to the gravel mines near Lagrange and se at Patricksville, about 4 miles southwest from Lagrange. Very

little of its water has been used for any other purpose. The ditch may ultimately, however, come into service as an irrigation ditch as the demand for water for hydraulic mining decreases. The ditch is carried under the gravel deposit of Indian Bar in a box culvert; thence follows the south side canyon wall for 14 miles, being always within one-half mile of the river, and reaching the vicinity of Lagrange at an elevation of 250 to 300 feet above the water surface of the river. There are many and long flumes in use where the canyon walls are too steep and rocky to permit of open-ditch construction. The ditch has an average bottom width of about 6 feet and carries water 3 to 3½ feet deep. Its grade is 7 to 8 feet to the mile and its capacity 50 to 60 second-feet. The cost of this ditch was about $300,000. The total expenditure on the ditch property is, however, reported to have been $800,000. The annual cost of repair and maintenance is $3,000 to $4,000. The ditch drops its water into the Patricksville Reservoir about 3 miles south of Lagrange. The reservoir is formed by means of two earth dams. It has a surface area of 132 acres and a capacity of about 57,500,000 cubic feet, or 1,320 acre-feet, its average depth being about 10 feet.

Wheaton Ditch.—This ditch has gone out of service. It was a small ditch on the south side of the river, having its head at the Wheaton Dam, which was maintained in the mouth of the Tuolumne River Canyon, about 1½ miles above Lagrange, until replaced in 1891–1893 by the massive masonry structure of the Turlock and Modesto irrigation districts. Several water companies were early interested in diverting water for mining purposes from the river at or near the Wheaton Dam site, just above French Bar. Of these, the Franklin Water Company was incorporated in 1854. It and the French Bar Mining Company both sold out to the Stanislaus County Water Company in 1855. The purpose of the new company was to supply water to Lagrange and vicinity by means of ditches on each side of the river. Water was diverted from the river on the south side at a point about 500 feet above the falls or rapids, the latter being the point subsequently selected as the site for the dam. This water company sold out to the Tuolumne Water Company in 1871. The latter company at once constructed a dam, a substantial cribwork structure of sawed lumber. Its height ranged from 16 to 30 feet. Its lower face was nearly vertical. Its upstream face was covered with a timber skin, which was renewed but once during the entire life of the structure. The upstream face was but little flatter than 1 to 1. The crest length of the dam, not including the overfall wall forming the outer ditch bank at its head, was 125 feet. About 250,000 feet of lumber were used in its construction. It is reported to have cost about $24,000. Several years after the construction of this dam the entire property of the water company was sold to Mr. M. A. Wheaton, one of the stockholders, and a small ditch about 2 miles long was kept in service. Its duty gradually dwindled to the irrigation of a few acres of orchard and vegetable land (not exceeding 50 acres) in the river bottoms just below Lagrange.

A.

B.

LAGRANGE DAM DURING CONSTRUCTION.

A, As seen from front; *B*, Abutment.

ange Dam.—The structure which replaced the Wheaton Dam is ly known as the Lagrange Dam. It was built by the Turlock desto irrigation districts. It is noteworthy as the highest over- n ever built on a large river. For the irrigation of the Turlock on district, on the south side of Tuolumne River, which was ed in 1887, it was necessary to construct a high-line canal. A am at or near the Wheaton Dam site was decided upon. After eliberation the Modesto irrigation district, on the north side of er, determined to join in the construction of the dam, as it ed desirable to have joint control of the diverting works with er district and because it afforded the best command of canal westward to the district lands. (See Pls. X and XI and fig. 12.)

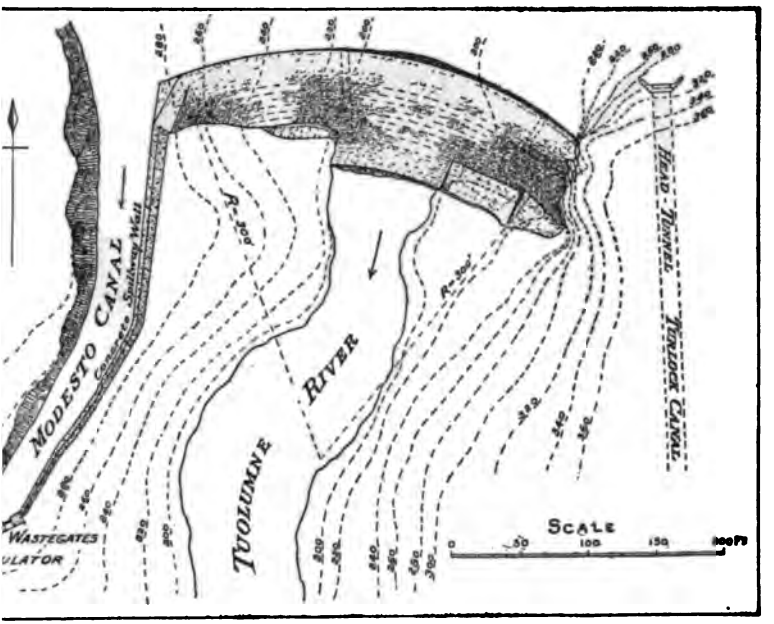


FIG. 12.—Plan of Lagrange Dam.

engineers of the two districts were required to prepare plans for ucture, from which those submitted by Mr. Luther Wagoner lopted; but as Mr. Wagoner soon afterwards resigned his posi- engineer for the Modesto district, Mr. E. H. Barton, the Turlock engineer, was placed in charge of construction. The dam from point of its foundation surface to its crest line has a height of t. It raises the surface of the river 100 feet. It is arched am, its upstream face being curved to a radius of 301 feet. Its ength from the retaining wall built for the outside bank of the o canal to the cliff on the south side of the river is 310 feet. stream face of the dam is vertical from its top downward for 70 *ance to the foundation surface* it has a batter of 1 in 20. The

top edges of the dam are rounded off toward the upstream face on a radius of 3 feet; toward the inclined downstream face on a radius of 17½ feet. The level portion of the top has a width of 6 feet. At 6 feet below its crest line the dam is 24.13 feet thick; at 69 feet below its top it is 52 feet thick; at 89 feet it is 66.25 feet; and at 97 feet, the top of the foundation masonry, it is 84 feet thick. The extreme bottom width at the highest point of the dam is 90 feet. The downstream face has a pitch of 1 on ½ (about 25°) down to 70 feet below the crest height of the structure, where a compound curve 64 and 23 feet radii commences, which carries the face to its intersection with the downstream face of the foundation masonry, about 3 feet above low water.

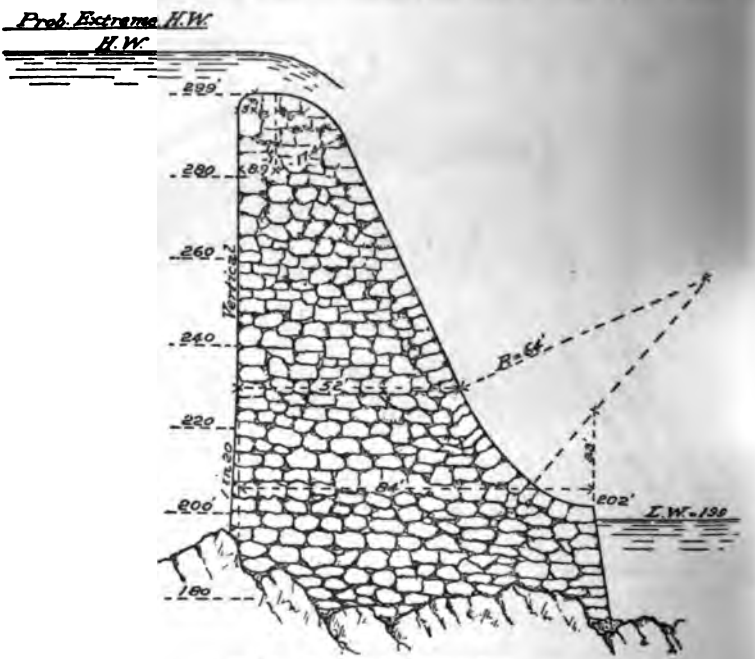


FIG. 13.—Section of Lagrange Dam.

From this line down to bed rock the masonry has a batter of 1 in 7. The dam thus affords a smooth support for the overfalling waters, which shoot off from the almost horizontal surface at the termination of the downstream face with great force against the river below. The original design contemplated the use of a second or subsidiary dam below the first, to create a pool of dead water, or water cushion, against which the force of the overfalling stream might be expended, but this has not been carried out. As the low-water width of the river—or rather its width at the height of top of the foundation—is only 80 feet, to be compared with 310, the crest length of the dam, it is apparent that during high stages of the river there must be a considerable crowding of the water from both sides toward the center of

A.

B.

LAGRANGE DAM.

A, Nearing completion; B, At time of high water.

ie stream. What effect this will have upon the solid rock walls of ie canyon is not yet known; but if these show any tendency to break way on crevice and cleavage lines recourse will be had to a reenforce-ient with cement mortar and concrete. The dam is constructed iroughout of uncoursed rubble laid in cement concrete. The cement sed in the structure was furnished by the irrigation districts. The ork of stripping and cleaning the foundation and putting masonry in lace was all covered by the price of $10.39 per cubic yard of masonry, t which construction was contracted for. The amount of masonry in ie dam is about 39,500 cubic yards. Nearly 30,000 barrels of Port-ind cement were used in its construction. The total cost of the dam, ot including canal head works, was about $550,000. This great cost f work executed under comparatively favorable conditions must be scribed largely to the lack of active competition among bidders for ie work. There was doubt concerning the ability of the two districts) meet their obligations in coin at the time of the letting, because onds were not selling readily at the prescribed limit of 90 per cent, nd the contractors were compelled to make arangements to receive at ast a portion of their pay in district bonds. During construction of ie foundation masonry in 1891 the river was carried past the work in flume. Two openings or culverts were left through the masonry at ie elevation of low water and one about 10 feet above low water. hese were 4 feet wide and 6 feet high, and easily passed the low-water ow of the river in the following seasons. The two lower ones were losed when the masonry had reached sufficient heigth to force the full ow-water flow through the upper culvert. The upper culvert remained pen long after the completion of the dam. It was closed in 1896. Vater went over the top of the finished dam for the first time on the iorning of December 23, 1893.

TURLOCK IRRIGATION DISTRICT.

This irrigation district was organized in April, 1887, under favorable uspices. There was but little opposition. Mr. George Manuel was iade district engineer and estimated the cost of the necessary works t $467,545. Bonds were issued in the amount of $600,000, and when t became apparent that this sum would not cover the cost of the works n additional issue of $600,000 was made. Pending protracted litiga-ion and the negotiations with the Modesto irrigation district relating o its participation in the construction of a high dam above Lagrange, he work on the main canal was pushed as rapidly as possible; but it is iot even yet completed.

This district lies on the south side of Tuolumne River, extending outhward to near Merced River and from the western edge of the Sierra Nevada foothills westward to near San Joaquin River. It has an area of 176,210 acres, of which about 30,000 are in Merced County, and the remainder *in Stanislaus County.* The land in the district is generally

held in tracts of less than 1,000 acres, practically all of which are grain farms. The soil is generally a sandy loam, fertile with moisture, producing fair crops of cereals when the rainfall reaches 10 inches favorably distributed through the season; but on the whole there has been little profit in wheat farming.

Water is to be taken from above the Lagrange Dam by a tunnel 5[illegible] feet long, 12 feet wide, and 11 feet high, with vertical sides, and a radius of arch of 6 feet. It was originally proposed to place the regulator and a waste gate at the lower end of this tunnel, but this idea was abandoned and a regulating gate has been constructed at its inlet. For over a mile below the head-works tunnel the canal winds along a slaty hillside. The canal has there been constructed with a bed width of 20 feet and a proposed water depth of 10 feet. The outside canal bank is constructed of dry-laid rock retaining walls with clay puddle core. At the lower end of this hillside work the canal crosses Snake Ravine, which was closed by a dam and up which the canal water is to be turned for about 1,000 feet. The earth dam has been washed out, and the pond which it was to form is to be replaced by a flume. Thence the course of the canal leads through a cut about 800 feet long, made by the hydraulic process, and its water is dropped into old mining pits, in which it is held for a distance of about 2,000 feet. This canal section is followed by a rock cut 860 feet long, leading into Dry Creek. The channel of Dry Creek is utilized for 1¼ miles, and this creek section of the canal is followed by a flume. In its course farther westward the canal pierces three foothill spurs in tunnels, none of which are lined. Below the last of these tunnels the canal crosses a gulch, which is to be closed with a concrete dam 40 feet high, and the canal then enters another cut 3,300 feet long. It will cross Peasley Creek in a flume 20 feet wide and 7 feet deep.

The canal below Peasley Creek is generally 45 feet wide on the bottom and 7½ feet deep, and has a fall of 1 foot to the mile; but dimensions are not uniform, being varied to suit the character of material into which the canal is cut. The canal capacity as reported by the district engineer is 1,500 second-feet. Great care has been taken to keep canal dimensions uniform and to bring the canal bed to grade, often necessitating the taking of material from borrow pits at the outer levee base, thereby increasing the apparent height of canal levees and greatly increasing the danger of breaks in these embankments. There would have been no objection, so far as simple inspection discloses, to deepening the canal below grade where necessary to secure material, and the cause for adopting the above type of cross section has not been made apparent. Drops or check gates have been constructed where material is liable to erosion and the natural fall exceeds that prescribed for the canal. The canal approaches some of these between high levees and leaves them in moderate cut—a questionable arrangement. It might even be better to bring the canal into service without the use of drops

d to place these later where required. The main canal is 18 miles g and will supply water to five principal laterals, which are to have an gregate length of 80 miles. These are to be 30 feet wide on the bottom d 5 feet deep, and are to have an effective fall of 2 feet to the mile. her branch ditches of smaller dimensions, to the extent of 180 miles aggregate length, are also to be constructed by the district. As the face of the district is very smooth and the fall quite uniform from theast to southwest, most of the branch canals will be constructed land lines from north to south and from east to west. It is underod that a contract for the completion of this entire system of ditches, luding the main canal, has been let, and that work is to be actively shed under this contract.[1]

MODESTO IRRIGATION DISTRICT.

oon after the project of organizing the Turlock district had taken inite shape, the farmers on the north side of Tuolumne River olved upon the formation of a second district to be dependent on t stream for its water. Although considerable opposition was ountered, the result of their action was the formation of the Moto irrigation district, which was consummated in July, 1887. The trict extended northward from Tuolumne River to Stanislaus River, l from the foothills westward into the trough of the valley to near n Joaquin River. It had an area of about 108,000 acres. The preinary study indicated conclusively that the expense of constructing anal from Tuolumne River to command the entire district would be hibitory, and that either the high southeastern portion of the ginal district must be excluded or that water must be taken from nislaus River at an estimated cost of over $700,000. Before further ps were taken bonds to the amount of $800,000 were voted, and a luction of the area of the district to 81,500 acres soon followed. The ject for the irrigation of the district lands did not assume its final inite shape until an arrangement was entered into with the Turlock igation district, according to which the two districts were to unite constructing a great diverting dam in Tuolumne River about 1½ les above Lagrange, and agreed to contribute equally toward its nstruction.

The canal has an inlet capacity of about 750 second-feet. Now hang- g on hillsides, now following table-lands or mesas, or penetrating low ars of the foothills in tunnels, it reaches a point near Waterford, th a total length of 23 miles. The canal regulator or head gate is place and main canal excavation is reported finished, but other ructures are incomplete or wanting, such as check weirs, flumes along eep hillsides and over creeks and gulches, and road bridges. This, cluding one-half the cost of the Lagrange Dam, has been accom-

[1] For further description of this canal, see American irrigation engineering, by H. M. Wilson, Thirteenth Ann. Rept. U. S. Geol. Survey, Part III, 1893, pp. 203, 232.

plished, under district management, with an expenditure, expressed in bonds sold, of the entire first issue of $800,000. The result is hardly more satisfactory than in many other of the irrigation districts. The district is not yet in position to furnish water to any of its lands; the cost of the work has been excessive, owing largely to the uncertainty of pay on completion, which deterred contractors in the case of nearly all district work in this and other irrigation districts from making low bids. Several years ago additional bonds to the amount of $350,000 were voted by the district, but these are to be withheld from sale pending final settlement of matters that have involved the district in litigation.

The surface of the district is admirably adapted for cultivation by irrigation. The soils are sands and sandy loams, and the ground has a slope from east to west of 5 to 10 feet to the mile. The entire area of this district, as well as of the Turlock district, was originally a treeless plain. For many years past it has been a vast wheat field.

STANISLAUS RIVER.

HYDROGRAPHY.

Stanislaus River is of the same type as the Merced and the Tuolumne, and of equal rank as a source of supply for large irrigation canals. Its mountain watershed has an area of 1,051 square miles, the run-off from which is ordinarily greatest in May, when snows are melting. The stream is at a low stage from August to December. During the winter months its flow is variable, being greatly influenced by rainfall and temperature. Its mean monthly discharge increases rapidly from January to April. From April to June is the period of its spring rise. The low-water flow is ordinarily from 100 to 200 second-feet, and the mean flow during the spring rise is 3,000 to 5,000 second-feet.[1] At Oakdale, where the river enters San Joaquin Valley, its bed is depressed about 80 feet below the general surface of the main valley plain. The river's low banks are flanked on either side by narrow strips of bottom land, which extend from river to base of the slope rising to the higher level of the upland. For over 40 miles below Oakdale the river surface is thus depressed below the level of the valley lands, but the difference in level gradually decreases, and near the trough of the valley the bottom lands of the river are merged into the submersible portion of the east-side plain.

Irrigation from this river has been confined to the foothill region until the enlargement and extension of an old ditch within the last two years has made water available on the north-side upland.

The United States Geological Survey has maintained a gaging station on this river since June, 1895. It is located at the highway bridge

[1] See Physical Data and Statistics, W. H. Hall, State engineer of California, 1886, pp. 428, 476.

just above the bridge of the Southern Pacific Company at the town of Oakdale.[1]

The following facts are brought out from this record:

The low-water flow for 1895, occurring in August, September, and October, was 145 second-feet.

1896. Maximum discharge, May, 13,050 second-feet; minimum, January, 150 second-feet; average for the year, 1,935 second-feet.

1897. Maximum discharge, February 4, 10,580 second-feet; minimum, September and October, 140 second-feet; average for the year, 1,835 second-feet.

Estimated monthly discharge of Stanislaus River at Oakdale, California.

[Drainage area, 1,051 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1895.						
June	8,510	3,842	5,686	338,340	6.038	5,410
July	3,842	550	1,778	109,330	1.950	1.692
August	550	145	330	20,304	.362	0.314
September	3,161	145	638	37,970	.677	0.607
October	246	145	176	10,802	.193	0.167
November	179	179	179	10,651	.190	0.170
December	414	145	198	12,208	.218	0.189
1896.						
January	6,956	150	1,806	111,058	1.98	1.72
February	1,208	632	782	45,038	0.80	0.75
March	12,000	694	2,464	151,505	2.70	2.35
April	9,050	1,966	3,274	194,870	3.48	3.11
May	13,050	2,290	4,717	290,092	5.17	4.57
June	11,000	2,656	6,541	389,247	6.97	6.22
July	3,156	470	1,293	79,503	1.42	1.23
August	2,412	218	385	23,703	0.42	0.37
September	962	252	332	19,761	0.35	0.32
October	370	218	239	14,695	0.26	0.22
November	4,514	252	705	41,968	0.75	0.67
December	1,750	470	681	41,904	0.75	0.65
The year	13,050	150	1,935	1,403,344	25.05	1.85
1897.						
January	2,251	410	609	37,446	0.67	0.58
February	10,580	1,264	3,252	180,607	3.23	3.10
March	5,062	1,264	1,915	117,750	2.10	1.82
April	9,234	1,880	5,064	301,328	5.37	4.81
May	9,980	5,480	7,324	450,338	8.04	6.97
June	3,974	860	2,077	123,590	2.21	1.98
July	1,180	250	582	35,786	0.63	0.55
August	230	170	198	12,175	0.22	0.19
September	190	140	152	9,045	0.16	0.14
October	250	140	176	10,822	0.20	0.17
November	780	170	255	15,174	0.27	0.24
December	1,516	210	411	25,272	0.45	0.39
The year	10,580	140	1,835	1,319,333	23.55	1.74

[1] Bull. U. S. Geol. Survey No. 140, 1896, p. 305; Eighteenth Ann. Rept., Part IV, 1897, p. 371; Water-Supply and Irrigation Paper No. 11, 1897, p. 90; Water-Supply and Irrigation Paper No. 16, 1898, p. 187.

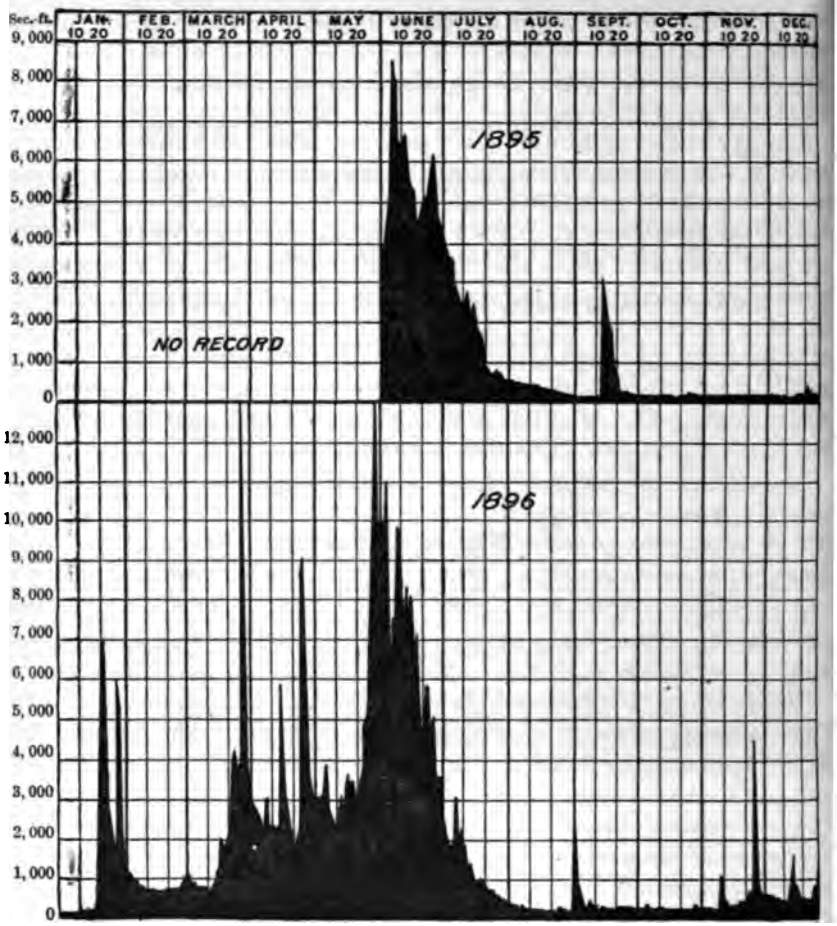


FIG. 14.—Discharge of Stanislaus River at Oakdale, California, 1895-96.

CANALS AND DITCHES.

Stanislaus & San Joaquin Canal.—Among the ditches constructed as aids to placer and river-bed mining over forty years ago was one on the north side of Stanislaus River, constructed by the San Joaquin County Water Company. Water was diverted from the river about 7 miles above Knights Ferry and was carried in ditch and flumes along the broken northern wall of the river canyon to points at and near Knights Ferry, where the ditch water had attained an elevation of 150 feet above the river surface. The ditch constructed by this company in 1854 to 1856 has remained in service ever since. As the demand for water from miners decreased, the demand from irrigators increased. Many projects for the diversion of a greater quantity of water than that represented by the old ditch capacity were proposed, but none

ushed to completion, until finally, under new management, an ment and extension of the old work was successfully accom- within the last few years, and is now known as the Stanislaus Joaquin Canal.

old ditch before enlargement had somewhat irregular dimen- It was generally 4 feet wide on the bottom, about 5 feet at the d carried water 2 feet deep. Its fall was not uniform, gen- about 5 feet to the mile. Its capacity was about 15 second-feet. was constructed across Stanislaus River at its head in several s, closing the spaces between high projecting points of bed rock. aces into which these divided the river bed were closed by means er crib work having a crest length of about 300 feet and an e height of about 16 feet. It raises the water surface of the 2 to 14 feet. A portion of this dam was renewed in the years 1880.

north-bank end of the dam rests against a high timber crib filled ck, which is perforated for the ditch. The inflow into the ditch rolled by means of a gate operated with a winch. When it was l to increase the capacity of the original ditch, about 1888, work mmenced by a new organization, known as the San Joaquin and Water Company, on a masonry dam 4,300 feet below the t head of the ditch, but the structure has not been completed.

old ditch for nearly a mile below its head had a position below reme limit of the river's flood plane. It has therefore frequently it out or filled with sand during the river's high stages, and its have been put to much trouble in keeping this upper section At 3 or 4 miles below its head the ditch was carried for over eet along a smooth-surfaced vertical cliff in a hanging flume, iron supports were firmly anchored to the solid rock. The t Knights Landing was at such elevation that but a short tunnel cessary to carry a branch northward through the divide into Creek Valley. Its water was there used in one of the pioneer rds of the State, the property of Judge Schell. The original the old ditch was $80,000. At the time it was superseded by w work its waters were being used on 200 to 300 acres of orchard, rd, and garden land.

matter of history, reference may be made in this connection to rmington Water Company, whose works were intended to sup- hose of the older ditch. This company, with Colonel Bulkley as ngineer, proposed to construct a loose-rock dam across Stanislaus one-quarter mile above the head of the San Joaquin County ditch turn the river through a rock cut past the south end of the dam. cation selected as a dam site was a favorable one, but the route he dam for a large canal for 2 or 3 miles to the point where it roposed to construct a tunnel through the ridge between the nd Little John Creek was not favorable. The construction of the

tunnel was, however, commenced, and it was driven into the mountain 700 feet from its south end and 200 feet from its north end. The length of the finished tunnel would have been 1,940 feet. This work was done in 1874 and 1875, but was not carried to completion, because the necessary financial arrangements could not be made.

The successive failures to accomplish a diversion of Stanislaus River water in a large north-side ditch, including a number that did not progress far beyond the recording of a claim to water, did not deter the Stanislaus and San Joaquin Water Company from undertaking the work.

The canal as it came into service in 1896 received water through an upper ditch extending from the diverting dam to the proposed new dam, 8 feet wide on the bottom, sides nearly vertical, water depth 5 feet, and a fall of a little over 4 feet to the mile. Below the partially built new dam the ditch for a distance of 900 feet is 9 feet wide on the bottom and its water depth was increased to 6 feet. The canal then enters a flume 1,112 feet long, thence flows through a tunnel 1,047 feet long, thence for 1,520 feet in another flume, from which it is delivered into a second tunnel 1,037 feet in length, and thence by ditch and flumes to a point to the north of Knights Ferry, where the Farmington Branch is sent off practically on the line of the old Schell Ditch to Little John Creek.

The main ditch is carried from there in a flume half a mile long across the depression to the north of Knights Ferry, thence by short ditch into a natural channel, in which it flows westward several miles, thence again in ditch and flumes alternately to a point about 11 miles west of Knights Ferry, where a distribution of its water is to be made to three branch ditches or laterals; of these one, the Escalon lateral, is to supply water to the uplands near Stanislaus River; a second, occupying the natural channel known as Lone Tree Slough and continued as Lathrop lateral, will command the main body of land intermediate between French Camp Creek and Stanislaus River; and the third, taking a northwesterly course, is to skirt the foothill base. From the main ditch a flume, in part supported by a bridge, has been carried across Stanislaus River for the delivery to Oakdale of 80 second-feet of water.

The Farmington Branch flows in Little John Creek about 11 miles, thence in a ditch on the north side of the creek 12 miles, reaching and extending upon the valley plain to the northwest of Farmington. It is 8 feet wide on the bottom, will carry water 2 feet deep, and has a fall of 1½ feet to the mile. The Escalon and Lathrop laterals are 14 and 24 feet wide, respectively. They were not complete in 1896, neither had the proposed Clyde lateral been constructed.

The upper two flumes on the main canal are 10 feet wide by 6 feet deep. These are followed by three flumes 8 feet wide by 8 feet deep. The gradient of main flumes varies from 1.9 to nearly 2.5 feet in 1,000.

The flumes below Knights Ferry generally are 8 feet wide and 6 feet deep, on a grade of 1.6 feet in 1,000; but the long flume is an exception; it is 6 feet wide, 5 feet deep, and has a fall of 11.4 feet in 1,000. The upper tunnel is 12 feet wide, planned to carry water 6 feet deep, on a grade of 1.5 in 1,000. The second tunnel is 14 feet wide, but has a grade of only 1.1 in 1,000. Where the upper portion of the main ditch is in excavation it is an enlargement of the old ditch. The outer bank of the old ditch was not disturbed, the entire enlargement having been effected by deepening and by cutting into the hillside. The new ditch is 9 feet wide on the bottom; it is to carry a maximum depth of 6 feet of water, and its fall is 0.9 foot in 1,000.

The great variability in capacity which the above dimensions indicate is to be accounted for in a measure by the fact that further enlargement is contemplated and that the new work was fitted to the old, as circumstances permitted. The total expense of canal enlargement, not including engineering nor rights of way, was about $150,000.[1]

Water is being furnished to irrigators very nearly on the same plan as that adopted by the Fresno and Crocker-Huffman companies, and long before water was available for delivery contracts were entered into between farmers and the canal company according to which the landowner pledged himself to take water for a specified tract of land as soon as water was available, and to pay therefor a bonus of $10 per acre, in deferred payments, at the rate of 50 cents per acre per annum, with interest at 6 per cent per annum on the amount remaining unpaid. The takers of water rights, in addition to the payment of this bonus, also obligate themselves to pay an annual water rental at the rate of $1.50 per acre. But the rate per acre is to become due during the first four years on only so much of the land as is actually irrigated; on not less, however, than one-fourth of the entire area the first year after water is available, on one-half the second year, on three-fourths the third year, and on the full acreage the fourth year and thereafter. The water rights issued at the beginning of 1896 covered about 14,000 acres. No information is available as to extent of irrigation during the first season.

Knights Ferry & Table Mountain Ditch (also known as *Pentland Ditch*).—This ditch, which has been out of service since 1881, is mentioned because before its final abandonment it supplied water to a few acres of orchard, vineyard, and garden land on the south side of Stanislaus River, opposite Knights Ferry. This ditch was constructed for mining purposes in 1855, at a reported cost of $210,000. Its head was at Sixmile Bar, about a mile below the head of the San Joaquin County Ditch. At that point a log dam, 30 feet in maximum height, enabled diversion into the ditch, which was cut into the south canyon wall. The location was not favorable for a ditch; flumes had to be used at a number of points, and at others the ditch was supported by

[1] The above facts relative to the work done by the new water company are largely from the published report on the work by Prof. George Davidson.

a high outside wall of dry rubble. The flumes were 4½ feet wide and 2 feet deep. The ditch was about 8 miles long; it had a fall of about 8 feet to the mile. Its capacity was about 20 second-feet.

Oakdale Canal.—From a point on the south side of Stanislaus River about a mile below Knights Ferry, a canal has been constructed to Oakdale. It has a total length of about 12 miles, and commands the low mesa, or second bottom, on which the town of Oakdale is situated. The construction of this canal was commenced about 1888, but much difficulty was experienced in making the diversion. At its head a loose-rock dam was to be maintained in the river. This was constructed of large blocks of rock, which were abundant near the dam site, and it was expected that, if piled into the river in sufficient quantity, they would form a riffle or sill above which water would be at sufficient elevation to be forced into the canal. The low-water plane of the river was to be raised by the structure about 8 feet. Seven years after the first construction of this dam the river was still annually breaking through it, and arrangements were then made with the Stanislaus and San Joaquin Water Company to take water to the extent of 80 second-feet from the canal of that company. This water is delivered to the Oakdale Canal at a point some distance above Oakdale, through a flume which spans the river.

MOKELUMNE RIVER.

HYDROGRAPHY.

Mokelumne River is the most northerly of the Sierra Nevada streams which are directly tributary to San Joaquin River. Its mountain drainage basin has an area of 657 square miles. In character it does not differ materially from the Stanislaus, the Tuolumne, and the Merced. Its bottom lands are nearly as extensive as those of Merced River, but do not lie in so compact a body. They extend well into the foothill region and westward to near Woodbridge. At that point there is a gradual merging of the upland and river bottom, this point being at the eastern limit of lands naturally subject to inundation.

The river is subject to the usual freshet flow of all Sierra Nevada rivers in winter, and has a pronounced high stage late in spring, due to the melting of snow. Its flow in April, May, and June is generally 3,000 to 4,000 second-feet. At its lowest stage, in the fall of the year, it brings less than 100 second-feet of water into the main valley.[1] The lesser drainage area of this river when compared with the others of its class is offset in a measure by greater rainfall throughout its watershed.

The Southern Pacific Company commenced observations of gage heights on this river in 1879 at their bridge at Lodi. The United States Geological Survey made a number of measurements of discharge

[1] *See Physical Data and Statistics*, W. H. Hall, State engineer of California, pp. 422, 476.

at that point, but, owing to the poor character of the section, results were not altogether satisfactory and the station was discontinued. For results of the work at this point see various reports of the Survey.[1]

CANALS AND PROJECTS.

As most of the water of this river flows idly to the sea, it is not without interest to inquire why the water is not utilized, particularly as the upland on both sides of the stream is a friable sandy loam of excellent quality, well adapted for irrigation.

The rainfall of this portion of San Joaquin Valley is about 15 to 20 inches per annum, enough to permit cultivation of cereals by methods of dry farming, and even to make orchards and vineyards yield a fair return. The demand for water has never, therefore, been very imperative. On the other hand, many projects have been discussed or partially executed, but, on account of engineering and financial difficulties, have met with entire or partial failure. The most notable project was that of the Mokelumne Ditch and Irrigation Company, which, under its original organization, commenced but did not finish the construction of a high masonry overfall dam or weir at the Westmoreland bridge near Lancha Plana. After many years of inaction work was again undertaken in 1887. The dam was then raised and repaired and work on the canal was commenced. It was proposed to carry water in a large canal on the south side of the river along and through the hills into Bear Creek, and thence to the eastern border of the valley lands, where it was to be put into a system of branch ditches or canals. The project failed for lack of funds.

A few years later the farmers living near the eastern margin of the flood limit of the valley, between Stockton and Mokelumne River, gave sufficient encouragement to the Woodbridge Canal Company to warrant the construction of their proposed canal. The river was brought under control and raised at Woodbridge by a series of gates, equivalent to a movable dam, from above which a canal was cut toward the south. The canal came into use in 1891, but the demand for water was not so great as had been anticipated. Financial difficulties increased from year to year, and accidents happened to the head works, so that this canal also is again out of service. The irrigation with water from the river, therefore, is practically confined to a few acres of bottom land served by small neighborhood ditches, such as the Bahmert Ditch, on the north side of the river some miles above Clement. The principal mountain ditches which are fed by the river and its tributaries are still in service for mining and domestic use rather than for irrigation.

[1] Twelfth Ann. Rept. U. S. Geol. Survey, Part II, 1891, p. 323, Pl. LXXXVII; Bull. No. 131, 1895, p. 86; Bull. No. 140, 1896, p. 308.

WEST-SIDE CREEKS AND OTHER STREAMS OF SAN JOAQUIN VALLEY.

From this account of irrigation in San Joaquin Valley it will appe that here, as well as in the State at large, irrigation is a local rath than a general aid to farming operations, and that ample opportunit remains for further development. This is particularly true of the mo northerly portion of the valley, and applies with increasing force t Sacramento Valley, where irrigation has thus far barely secured a foo hold. It remains to be said, however, that the foregoing chapters ar not to be considered exhaustive, because neither time nor means a command permitted a revisit to the entire field, and little or no attentio has been given to the lesser irrigation work from small creeks inter mediate between the more important streams.

On the west side of the valley are a number of small irrigation work which draw their supply from the creeks of the Coast Range. These streams are torrential in character. Their flow is occasionally consider- able during severe storms, but it is of brief duration. Ordinarily th run-off is very slight, and in many winters the streams remain almos unaffected by the light fall of rain on the eastern slopes of the Coas Range. This uncertainty of supply interferes with attempts to brin them under control for irrigation purposes. Here and there, howeve some success has been had, particularly on those creeks having th largest drainage basins. This has been the case on Los Gatos Cree just above Huron, which has a drainage area of 441 square miles. Th bed of this creek some miles above Huron remains moist during sun mer, and diverting works there constructed (in part at least of sheet pi ing, nearly equivalent to a submerged dam) have developed a flow of to 8 second-feet for the Pleasant Valley Ditch, which is a private dit in use for the irrigation of a tract of alfalfa.

The favorable indications near the head of this ditch for the develo ment of a larger water supply encouraged the farmers to the east Huron to form an irrigation district, which was organized in 1891. is known as the Huron irrigation district, and has an area of abo 20,000 acres. A plan of works was adopted, but no bonds were sol and no construction work has been done.

Notable among great works projected but not carried out was t west-side canal, which was to have been fed with water from Tula Lake, it having been assumed at the time the project was under di cussion, in 1876 and 1877, that the lake could be relied on to overflo annually. As there has been no such overflow for about twenty year it appears fortunate that the project was not undertaken.

The subject of irrigation and water development from artesian wel should not be entirely neglected in treating of irrigation in San Joaqu Valley, but space will not permit of its consideration in this pape For information on the subject reference may be made to the speci bulletin of the Eleventh Census, Report on Agriculture by Irrigati *in the Western Part of* the United States, by F. H. Newell, page

INDEX.

Page.

Adobe Ranch Ditch, description of 31
Adobe Slough, figure showing head of ... 30
Blyth Canal, plate showing 24
description of 25
Buchanan Reservoir project, description of 32-33
Chowchilla Canal, description of 15-18
plan and section of regulator of 16
Chowchilla River, hydrography of 31
irrigation from 31-33
Cost of irrigation 21, 22, 29-32, 36
Crocker-Huffman Canal, description of .. 34-37
plates showing features of 34-36
Dean Ditch, description of 38
Dos Palos Canal, description of 24
East Side Canal, description of 25
Farmers Canal, description of 34
Feldhaus & Reed Ditch, description of .. 39
Fresno River, hydrography of 26
irrigation from 27-31
Fresno River Dam and Canal Company, operations of 27-30
figure showing canal of 27
Griffith & Shaver Ditch, description of .. 39
Hall, W. H., cited 10, 15, 17, 33, 40, 50
Hamlin Ditch, description of 39
Herndon, stream measurements at ... 10-11, 12
Ingalsbe Slough, description of 38
Irrigation, acreage under 20, 29, 37
cost of 21, 22, 29-32, 36
methods of 22, 29, 32, 40
Knights Ferry & Table Mountain Ditch, description of 55-56
Kohler, West & Minturn Canal, description of 31-32
figure showing water diversion by ... 32
Lagrange, stream measurements at 42-43
Lagrange Dam, plates showing 44, 46
description of 45-47
figures showing plan and section of .. 45, 46
Lagrange Ditch, description of 43-44
Madera Canal, description of 27-30
figure showing head of 27
Madera Canal and Irrigating Company, operations of 30
Madera irrigation district, work in 14-15
Means Ditch, description of 39
Merced Canal, description of 34-37
Merced Falls, water power utilized at ... 37
Merced Falls Ditch, description of 37
Merced Reservoir, description of 35
plate showing inlet to 38
Merced River, hydrography of 33
irrigation from 34-40
plate showing dam across 34
Miller & Lux, irrigation by 20-22, 24
Millrace Ditch, description of 39
Minturn, irrigation near 31
Modesto, stream measurements at 41-42

Page.

Modesto irrigation district, work in ... 45, 49-50
Mokelumne Ditch and Irrigation Company, operations of 57
Mokelumne River, hydrography of 56-57
irrigation from 57
Montgomery & Anderson Ditch, description of 38
Montgomery Ditch, description of 39
Murray Mill Ditch, description of 37-38
Newell, F. H., letter of transmittal by ... 7
cited 58
Oakdale, stream measurements at 51-52
Oakdale Canal, description of 56
Old River, irrigation from 38-39
Patricksville Reservoir, description of ... 44
Pentland Ditch, description of 55-56
Pleasant Valley Ditch, description of 58
Posey Slough, description of 24
San Joaquin & Kings River Canal, description of 18-23
figure showing head of 18
figure showing head works of 19
plate showing regulator in 22
San Joaquin County Water Company, operations of 52
San Joaquin Land and Water Company, operations of 53
San Joaquin River, hydrography of 9-12
diagram showing discharge of 12
irrigation from 13-26
figure showing dam across 20
plate showing steamboat chute on ... 20
San Joaquin Valley, map of 32
irrigation from west-side creeks of .. 58
Sanjon de Santa Rita, description of 24
Schuyler, J. D., quoted 15-16, 21-22
Scott Ditch, description of 38
Sillman & Ruddle Ditch, description of .. 39
Snelling Ditch, description of 38
Stanislaus & San Joaquin Canal, description of 52-55
Stanislaus River, hydrography of 50-52
irrigation from 52-56
diagram showing discharge of 52
Stevenson-Mitchell Canal, description of . 25
Temple Slough, description of 24
Tuolumne River, hydrography of 40-43
diagram showing discharge of 42, 43
irrigation from 43-50
Turlock irrigation district, work in ... 45, 47-49
Upper San Joaquin River Canal, description of 13-14
figure showing head of 13
Water, methods of diversion and use of .. 39-40
Wheaton Ditch, description of 44
Wilson, H. M., cited 33, 49
Woodbridge Canal Company, operations of 53

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 20

UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

ERIMENTS WITH WINDMILLS

BY

THOMAS O. PERRY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

CONTENTS.

	Page.
tter of transmittal	9
roduction by F. H. Newell	11
neral statement	19
Apparatus	21
Angle of weather	22
Artificial wind	22
Dynamometer	23
Method of experiment	24
Starting forces	25
Journal friction	25
Comparative standard wheel	27
ults of original tests	28
Tables and descriptions of wind wheels	28
Wheel No. 1	28
Wheel No. 2	29
Wheel No. 3	31
Wheel No. 4	33
Wheel No. 5	34
Wheel No. 6	34
Wheel No. 7	36
Wheel No. 8	37
Wheel No. 9	37
Wheel No. 10	38
Wheel No. 11	38
Wheel No. 12	39
Wheel No. 13	39
Wheel No. 14	40
Wheel No. 15	40
Wheel No. 16	41
Comparative results with plain sails	41
Wheel No. 17	43
Wheel No. 18	43
Wheel No. 19	44
Wheel No. 23	46
Comparison of several wheels	46
Wheels Nos. 2 and 50	46
Wheels Nos. 18, 19, 20, 21, and 22	47
Wheels Nos. 24, 25, 26, 27, and 28	47
Wheels Nos. 29, 30, 31, 32, and 33	49
Wheels Nos. 19, 20, and 34	51
Wheels Nos. 35, 36, 37, and 38	51
Wheels Nos. 40, 41, 42, and 43	53
Wheels Nos. *40, 41, and 42*	55
Wheels Nos. 35, 39, 40, and 44	57

Results of original tests—Continued. Page.
Comparison of several wheels—Continued.
Wheels Nos. 45, 46, 47, 48, and 49 59
Wheels Nos. 2, 60, and 61 61
Wheels Nos. 2, 49, 53, 54, 55, 56, and 57 64
Wheels Nos. 2, 49, 51, 52, 58, and 59 65
Sails at best angles of weather 65
Relations of three different velocities of wind 66
Automatic brake adjuster 67
Theory of the action of the wind upon the sails of windmills 69
Discussion of results 72
Thermometric and barometric influences 72
Construction of tables 73
Extreme results 73
Influence of outside wind 74
Standard of comparison 74
Corrections for axle friction 75
Aerial friction 76
Relation between speed and load 76
Best speeds for wind wheels 78
Best loads for wind wheels 78
Conical deflector in center of wheels 79
Air-cushioned sail surfaces 79
Obstructions on the back of sails 79
Waste of power due to the arms of wind wheels 80
Twisted sails 81
Deflectors in front of sails 81
Obstructions in front and in the rear of wind wheels 82
Maximum products in different velocities of wind 82
Starting forces and loads at the maximum in different velocities of wind. 83
Speeds of unloaded wheels in different velocities of wind 83
Action of air on the sails of a wind wheel 84
Theoretical efficiency of the wind wheel 84
Theoretical useful effort of wind 87
Aerial resistance to motion 89
Best number of sails 90
Best relative areas of sail surface 90
Actual efficiency of the wind wheel 91
Work of variable wind with variable load 92
Regulation of wind wheels 92
Limitation to size of wind wheels on account of weight 93
Efficiency of wind wheels as affected by diameter 94
Height of towers 94
Multiplication of wind power 95
Power of a 12-foot wind wheel 95
Value of mathematical formulæ in experimental investigations 96
Index 97

ILLUSTRATIONS.

		Page.
PLATE I.	Windmill pumping water for irrigation	11
II.	Reservoir and garden	12
III.	Windmill and small reservoir	14
IV.	Small irrigation reservoir	14
V.	*A*, Homemade wind engine, as used on the Great Plains; *B*, The Defender windmill and water elevator	16
VI.	Elevation of apparatus used in wheel tests	20
VII.	Plan of apparatus used in wheel tests	20
VIII.	Plan and elevation of dynamometer	22
IX.	End views of dynamometer	24
X.	Windmill and reservoir	78
XI.	Windmill and reservoir	88
XII.	Windmill on low tower on the Great Plains	94
1.	Vertical section through reservoir and outlet	12
2.	Windmills and circular reservoir	13
3.	Steel windmill and tower carrying tank	14
4.	Irrigating ditch leading from earth reservoir filled by windmills	16
5.	Distribution of water to small beds	17
6.	Distribution system adapted to irregular land	17
7.	Elevation and section of wheel No. 2	30
8.	Outline of area not available	31
9.	Elevation and section of wheel No. 3	32
10.	Dimensions of sails of wheel No. 3	33
11.	Angles of weather of sails of wheel No. 3: *A*, Angle at inner ends of sails; *B*, Angle at outer ends of sails	33
12.	Elevation and section of wheel No. 6	35
13.	Dimensions of sails of wheel No. 6	36
14.	Dimensions of sails of wheel No. 18	44
15.	Elevation and section of wheel No. 19	45
16.	Elevation and section of wheel No. 24	48
17.	Elevation and section of wheel No. 29	50
18.	Elevation and section of wheel No. 35	52
19.	Dimensions of sails of wheels Nos. 35, 36, 37, and 38	53
20.	Elevation and section of wheel No. 40	54
21.	Dimensions of concave sails of wheels Nos. 40, 41, 42, and 43	55
22.	Elevation and section of wheel No. 39	56
23.	Dimensions of sails of wheel No. 39	57
24.	Elevation and section of wheel No. 44	58
25.	Elevation and section of wheel No. 48	60
26.	Dimensions of sails of wheels Nos. 45, 46, 47, 48, and 49	61

Page
FIG. 27. Elevation and section of wheel No. 60 62
28. Elevation and section of wheel No. 61 63
29. Dimensions of sails of wheel No. 61 with obstruction on back 64
30. Action of wind on a sail 70
31. Action of air on the sails of a wind wheel 84
32–35. Theoretical efficiency of the windmill 85
36–39. Theoretical useful effort of wind 87-88

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, July 15, 1898.

SIR: I have the honor to transmit herewith a manuscript entitled Experiments with Windmills, prepared by Mr. Thomas O. Perry. These experiments, as described by Mr. Perry, were carried on during the years 1882 and 1883 for the United States Wind Engine and Pump Company, of Batavia, Illinois. As a result of these experiments radical changes and improvements were made in the windmills. As a matter of business policy the company did not desire that the results of these tests should be made known for some years. After the expiration of a certain time, however, the data have been placed at the disposal of the public through the kindness of the officers of the company and the efforts of Mr. Perry. Although as the result of this work great changes have been made in windmills, many of the suggestions made have not yet been put into practice and may serve as a foundation for further work along this line. The importance of the windmill as a means of utilizing the water resources of a part of the country is so great that all available information on the subject should be diffused and brought to the attention of persons who can make use of the facts. I respectfully request, therefore, that this manuscript, with a brief introduction, be printed as one of the series of pamphlets on water supply and irrigation.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

WINDMILL PUMPING WATER FOR IRRIGATION

INTRODUCTION.

By F. H. NEWELL.

During the progress of investigation of the extent to which the arid lands can be reclaimed by irrigation and of the related question of the occurrence of water underground attention has been continually drawn to the practical methods of bringing the underground water to the surface. Throughout a great part of the arid and semiarid region there are localities where water can be obtained at a short distance from the surface. The amount, although not large in the aggregate when compared with the quantity in some notable stream or lake, is yet inexhaustible by the ordinary methods of pumping. If, therefore, this water which exists from 10 to 50 feet beneath the surface could be cheaply raised, it would be practicable to utilize for agriculture tracts which otherwise have little or no value.

The irrigation of 20 acres in the midst of a section or township of land is, figuratively speaking, a mere drop in the bucket; but the reclamation of this small area generally means the utilization of adjoining lands. If, for example, 20 acres of some forage crop like alfalfa is made possible, this will result in obtaining a considerable amount of winter feed used in the sustenance of a herd which can be pastured upon the surrounding dry land. The successful cultivation of this 20 acres may thus directly or indirectly support a family, and, with increased experience and adaptation to the surrounding conditions, the family may in turn give place to a rural community. Given the existence of sufficient water underground to irrigate the 20 acres, the first question is that of ways and means of bringing the water to the surface.

The force which is ever present, making itself persistently felt throughout the Great Plains region, is the wind, which blows almost continuously. It carries the dust before it, cuts out the traveled roads, carries away the fine earth of the tilled fields, and builds up a fine dress, almost everywhere to be found. The wind, which has so long been considered as an annoyance and mischief-maker, has sufficient strength to perform the work of bringing water to the surface, if only suitable means of directing its energy can be discovered.

The *windmill is the best-known* method of converting wind energy *into work. In one form or another* it has been used from times ante-

dating the dark ages. In the twelfth century windmills, bu by individuals or by communities, were common. Some of th were of enormous size. In the German type t building on which the mill was placed was co in such a manner as to turn on a post in orde the sails into the wind. In the Dutch form t ing was fixed, but the head of the mill could l into the wind. The most notable use of th mills was in Holland, where the land was d pumping water from behind the dikes into th 1391 the bishop of Utrecht, holding that th the whole province belonged exclusively to h to the convent at Windsheim express perm build a windmill wherever it was thought pr so doing he overruled a neighboring lord who that the wind in the district belonged to hi years later the city of Haarlem obtained le Albert, Count Palatine of the Rhine, to buil mill, using the wind of the country.[1]

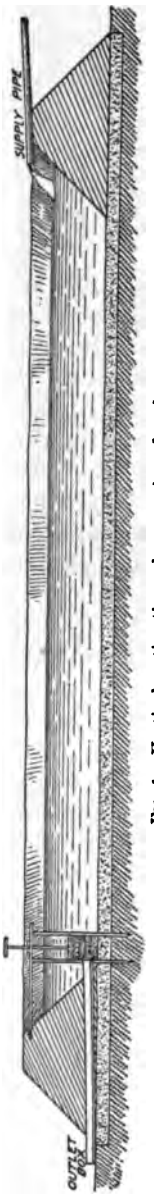
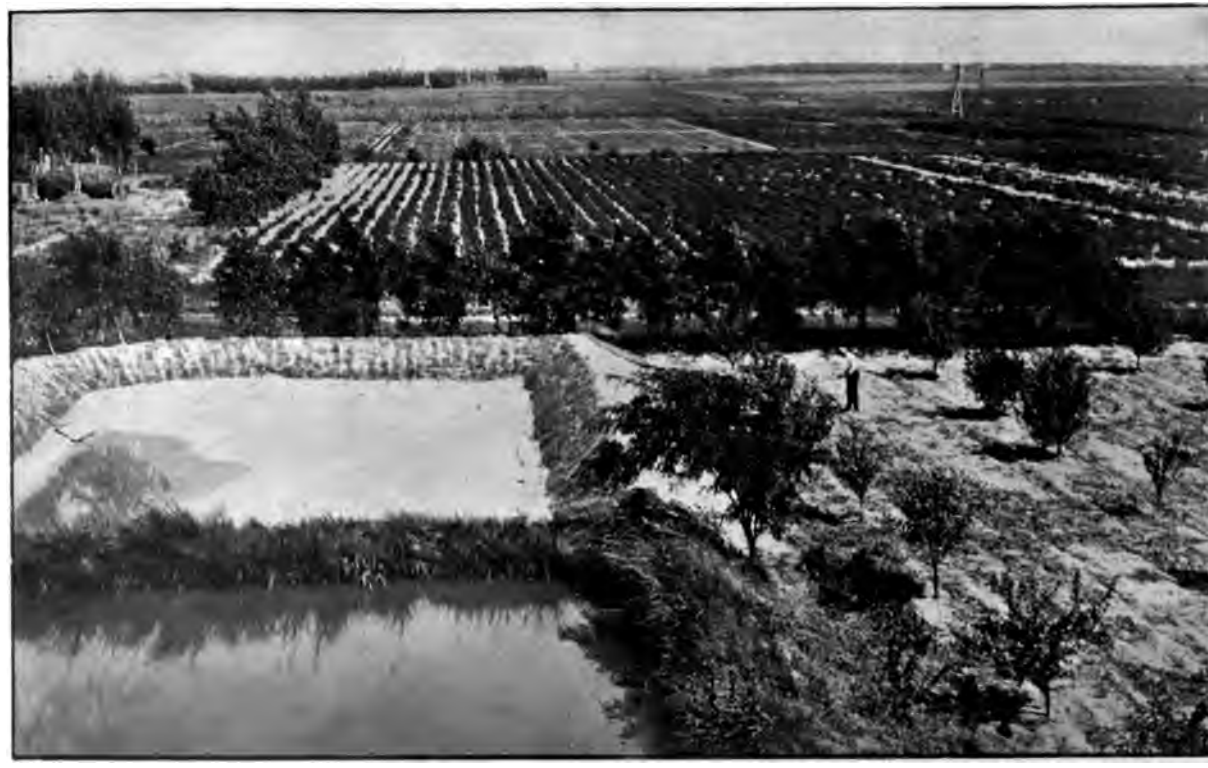


FIG. 1.—Vertical section through reservoir and outlet.

The huge, clumsy windmills of European n of which was erected at Lawrence, Kansas, w memory of the present inhabitants, have wit decades given place in this country to t rapidly running forms. Thousands of these h made by various firms throughout the cou first wood was used almost exclusively, bu being rapidly displaced by metal, especially steel plates and forgings. Although mi dollars have been invested in the manufac purchase of mills and much attention has be to the mechanical details and the saving in we cost, yet comparatively little study has been upon the actual efficiency of the various fo upon their development toward theoretical id

A view of gardens cultivated by water pu windmills is shown in the accompanying plat This picture has been taken from a windmill In the foreground is a small reservoir, divi bank in the middle, so that one part may be u pendently of the other. The part nearer the is the older; the second part is a recent addi dered necessary by the increase of the area cu Without windmills the cultivation of the tract try shown in this picture would be impossible. It is doubtful i cow could find subsistence on the area which now supports a

[1] *The Windmill as a Prime Mover*, by Alfred R. Wolff, p. 51.



RESERVOIR AND GARDEN.

Another small reservoir and windmill is shown on Pl. III. This reservoir is in the corner of a small suburban truck farm, to which it furnishes water both for vegetables and trees. The owner has stocked a small pond with fish and from it obtains an ample supply for his own use. On the right-hand side is shown the outlet gate, through which water is discharged into a ditch, and that in turn empties into furrows running near the fruit trees and traversing the cultivated ground.

In fig. 1 is given a section through one of these small reservoirs, showing at the bottom the puddled earth or clay that prevents the water from seeping into the adjacent ground. On this puddled earth the banks are built at a height of from 4 to 10 feet. These are usually built by plowing and scraping up the earth from the outside, the tramp-

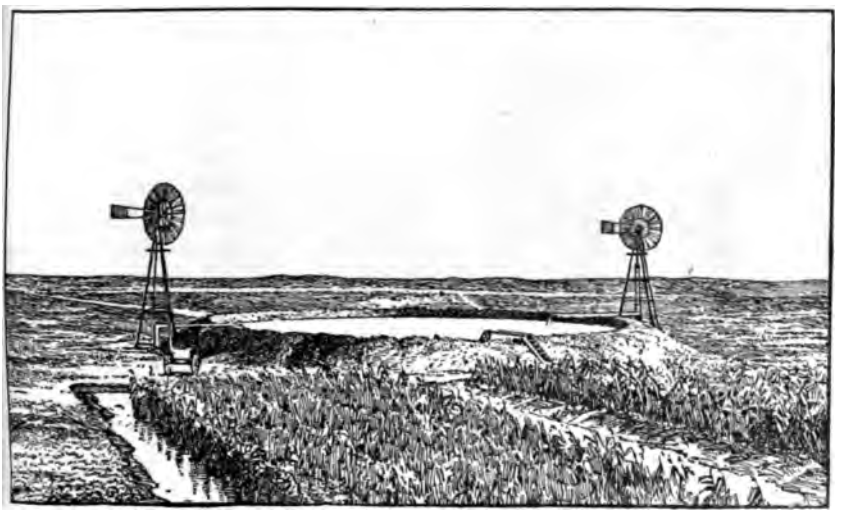


FIG. 2.—Windmills and circular reservoir.

ing of the horses and the men serving to consolidate it. When the bank has been built to the proper height it is smoothed and sodded. On the right-hand side of the figure is the pipe or wooden flume from the windmill, and on the left-hand side is shown the outlet box, which is usually built of 2-inch plank. This is closed by some simple form of wooden gate or valve, either lifted by means of a screw or hinged so as to open outward, and is held in place by the pressure of the water against it.

The square reservoir is the form usually adopted. The mills, as in the other cases, are placed on each side, pumping through short wooden flumes over the bank. These reservoirs are not only used for holding water for irrigation, but, as before stated, with a little care serve as ponds for raising fish.

A view of one of these small square reservoirs is shown on the accom-

panying plate (Pl. IV). The banks are obscured by the growth of weeds. The surface of the water is a little higher general level of the land, so that a supply can be drawn b directly to the adjacent fields.

Fig. 2 gives a nearer view of one of these earth reservoirs, this case has been built nearly circular in form. The two which su water are upon oppos in order pumps ma far apart as In many three or e mills, each erate size, a around a of consider The banks earth, are with sod t them from by the rai the waves times of hi

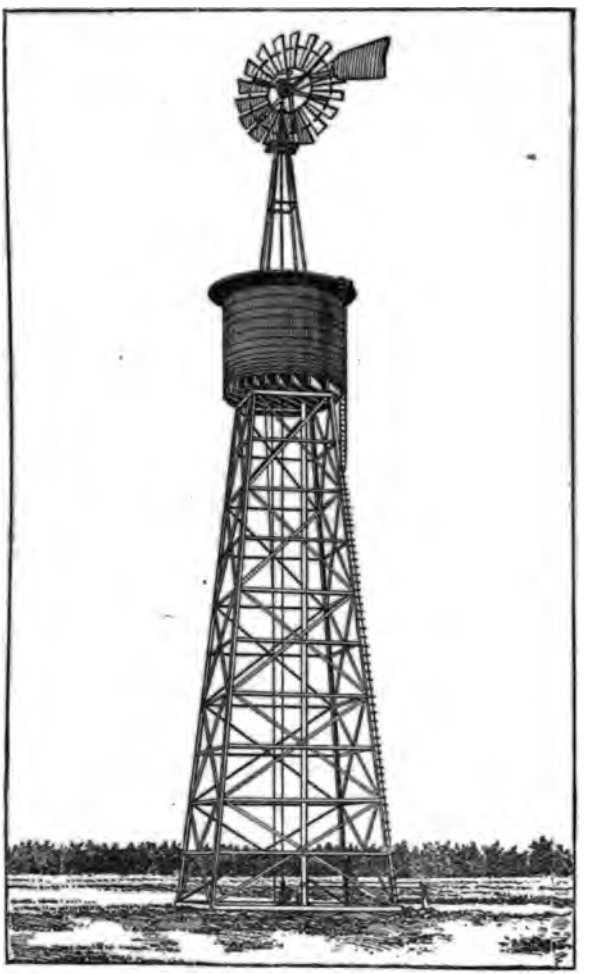


Fig. 3.—Steel windmill and tower carrying tank.

Fig. 3 r an adapta windmill f domestic w ply or for f water to or small to considerab ure. The gine is erec top of a h tower, whi supports a tank with cover to pr water from loss by evaporation. This device is generally emp railroad companies at stations on the Great Plains, where the be depended upon to force a sufficient supply into the tank f locomotives or for the railroad shops and offices. Many to depend for their water supply upon a windmill pumping wate *elevated tank, particularly* where the general surface is so ne

WINDMILL AND SMALL RESERVOIR.

SMALL IRRIGATION RESERVOIR.

at it is impossible to construct a small earth reservoir within reason-
le distance of the principal buildings.

The device shown in Pl. V, *A*, which has been employed to a consid-
able extent in the Great Plains region, is usually constructed by the
wners. The merit of the device is its cheapness. It may be built
ainly of old lumber and other material that can often be found about
e farm, such as axles or other gear from old farm machinery, bale
ire for staying the sails, and pieces of wood or metal which may
e classed as old junk. The machine can not be recommended on the
round of efficiency or economy. If a farmer has sufficient capital to
urchase and erect a good windmill he will undoubtedly succeed better
an by spending his time in making the cheaper device. On the other
and, in situations where, as is often the case in a dry region, the
rmer has lost crops year after year, has exhausted his resources, and
on the verge of bankruptcy, a contrivance of this kind may serve to
ave a small crop and give him a new start. In such instances there
sually will be found pieces of broken-down machinery about the farm.
ime and labor are commonly of little value where the ordinary farm-
ng operations have been unsuccessful, so that by the exercise of a
ittle ingenuity the material and energy that otherwise would be wasted
ay be turned to advantage.

The mill or engine consists of a shaft of wood or iron placed horizon-
ally and supported at each end. Upon this sails are fastened by arms
xtending out at right angles. On each end of the shaft is attached a
rank, and each of these cranks in turn drives some simple form of
omemade pump. The lower half of the mill is boxed in, and thus
rms a small building without roof, above which project the arms car-
ring the sails.

As illustrating another homemade device, Pl. V, *B*, has been intro-
iced. This mill and water elevator, invented by the owner, has been
ccessfully used to furnish water for irrigation, and, although not by
y means an economical device, nor one that can be recommended, it
as served its purpose. In other words, while as a rule it is economical
purchase the best, there are circumstances and times when for
ecial reasons the best mill can not be had, but it is still practicable
construct a machine which will accomplish the desired end, that of
tting water from the ground upon the land.

These examples might be almost indefinitely multiplied, but are suf-
cient to demonstrate the principle that with energy and ingenuity a
art toward irrigation can be made. When, however, some experi-
ice has been had in irrigation and newer mills are being procured, it
highly essential for continued success that something better than
e ordinary form of mill be obtained. Many of these have been
esigned for some other purpose than that of raising large quantities
f water through a short distance for irrigation. Some, for example,
ave been built with the idea of pumping a small quantity from great

depth for watering stock. Such mills, as a rule, do not fill the ı ments of the irrigator. Thousands of windmills are in use an sands more will be purchased, involving expenditures on the farmers aggregating millions of dollars. A saving of even ; percentage in cost and economy is therefore a matter of consi importance in the continued development of the water resource: country.

Fig. 4 shows two windmills so arranged as to pump into a re built of earth, placed upon the highest part of a farm. Runnin and day the pumps supply such a quantity of water to the re that when needed a considerable volume can be drawn at once

FIG. 4.—Irrigating ditch leading from earth reservoir filled by windmills.

a ditch of as large size as is ordinarily built for irrigating a from 10 to 20 acres.

Fig. 5 illustrates one of the methods used to distribute small ties of water, such as may be had from the tank or reservoir of ; mill. In the drawing the banks are represented as made of cer of wooden planks, such as are used in portions of California, water has especial value. The usual practice, however, witl Great Plains region is to make the ditches of earth and to pl little gates in wooden frames. Sometimes instead of using the irrigator controls the water by shoveling earth into the ditcl furrows, backing up the water until it overflows the bank or cor through on opening made by means of the shovel. As shown figure, the water flowing in the main stream from right to *checked by small gates* and forced to flow laterally through f

A. HOMEMADE WIND ENGINE, AS USED ON THE GREAT PLAINS.

B DEFENDER WINDMILL AND WATER ELEVATOR

is again checked in these and forced out upon the beds lying on ı side of the furrow. When a certain portion is wet the small ɔs are adjusted so as to force the water over other portions.

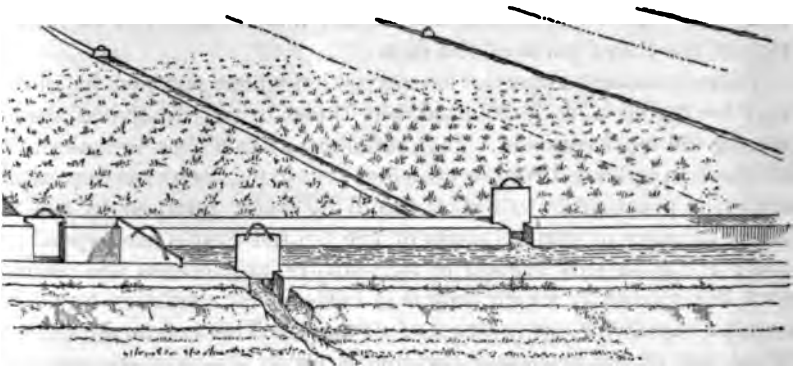


FIG. 5.—Distribution of water to small beds.

Where land has a decided fall or is rolling and it is too expensive to race, a system of distributing the water must be devised suited to the ıtour of the ground. In the case shown by the accompanying figure ;. 6) the water enters the distributing ditches at the upper left-hand

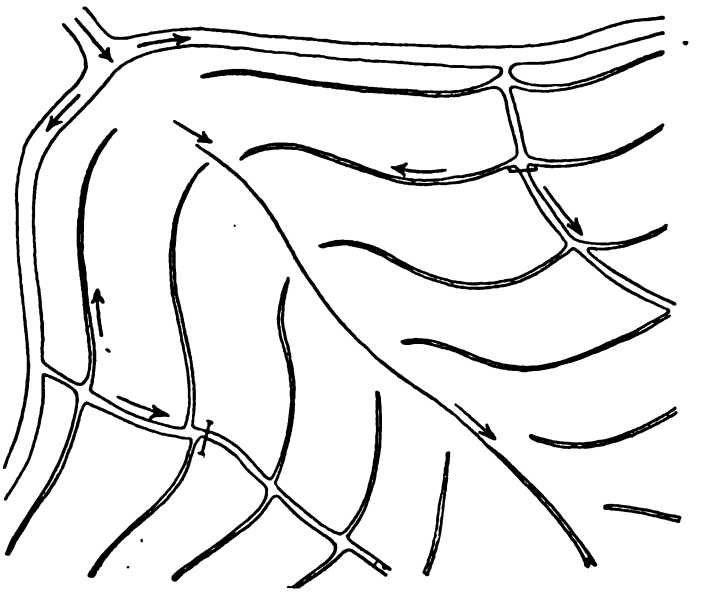


FIG. 6.—Distribution system adapted to irregular land.

ıer and, dividing, flows through these and again into still smaller hes, from which it is turned laterally into furrows. The flow is ɔked *from point to point by little* dams or temporary obstructions of

earth. After the water passes out upon a field any surplus is caugh by a small trench shown in the figure as trending diagonally towar the right-hand lower corner, and from this in turn the water can b caused to flow out upon the lower fields, so that the excess or seepag from the higher portions is not wasted but is employed in the irrig tion of the lower parts of the farm.

There are many other devices by which water pumped by windmil may be stored and distributed, but those briefly described above a among the most common. It is sufficient at this time to make mentio of these in order to bring out the fact that experiment has demonstrated that it is practicable to irrigate land by water pumped by windmills Many farmers in various parts of the United States are dependent fo their living on this system of agriculture, and others are seeking t gain knowledge by which they may better their condition by providin a water supply for use in times of drought. This is true not only of th West, but of the East and the South, especially in regions where th soil, though fertile, is light and becomes dry if the rains do not occu with great regularity. Even on the Atlantic seaboard the early truc farmers are experimenting with devices of this character, copyin methods which have already proved successful in Kansas. Modific tions in mills and other machinery are doubtless necessary to suit loc conditions. For a given purpose one mill may be far better tha another, and of two wind engines similar in cost one may be actuall worth two or three times as much as the other.

The dynamometric experiments carried on by Mr. Perry, as describe in the following pages, have brought out many important points upo which depend the relative efficiency of various mills and the advantage of one type over another. Some of these points have already been co sidered and made use of in mills now on the market, while others c equal or greater importance are awaiting practical application an ingenuity. It is hoped that by the publication of these facts interes will be aroused and inventors will be stimulated to continue work alon various lines, leading to further improvement and to a consequen cheapening of the cost of raising water by wind power.

EXPERIMENTS WITH WINDMILLS.

By THOMAS O. PERRY.

GENERAL STATEMENT.

The experiments with wind wheels described in the following pages were commenced June 1, 1882, and concluded September 15, 1883. At that time wind wheels in this country were nearly all made with narrow wooden slats for sails, set at various angles with the plane of the wheel, ranging from 35 to 45 degrees.

The slats were usually placed as close together as possible without having their projections on the plane of the wheel overlap. The proportions of sail surface and their angles of weather were apparently arrived at without any well-defined purpose. The only experiments made in the United States, so far as could be learned, related to starting forces only. They did not include the measurement of work in foot-pounds. The only well-defined experiments with wind wheels, the records of which were available, were those made in England by John Smeaton, F. R. S., about one hundred and twenty-five years before. Smeaton's paper, On the Construction and Effects of Windmill Sails, was read before the Royal Society May 31 and June 14, 1759, and to this day has remained the only definite available source of information on the subject treated. The paper was republished in Tredgold's Tracts on Hydraulics, a copy of which was received before these experiments were commenced.

It had been especially noted that Smeaton's angles of weather were much less than those in common use in this country, and note had also been taken of his statement to the effect that the work of a wind wheel is not increased, but diminished, by crowding it with sails so that their total surface exceeds about seven-eighths of the circular area containing them. The universal practice here was to crowd the wheel with slats until the total sail surface exceeded the total area of the annular zone containing them by more than one-fifth of the whole zone; according to Smeaton, this was adding to the greatest possible effective sail area more than 39 per cent additional area of worse than useless surface. If a large part of the material could be saved, and at the same time a considerable increase in power be effected, no slight

gain would be secured. Smeaton's experiments were made so long ago that they were disregarded, if they were ever noticed, by the builders of modern wind wheels. Our experiments fully confirmed Smeaton's results in regard to the greatest effective amount of sail area, although we do not necessarily accept Smeaton's judgment that it is not profitable to increase the total area of sails beyond about 37 per cent of the annular zone containing them.

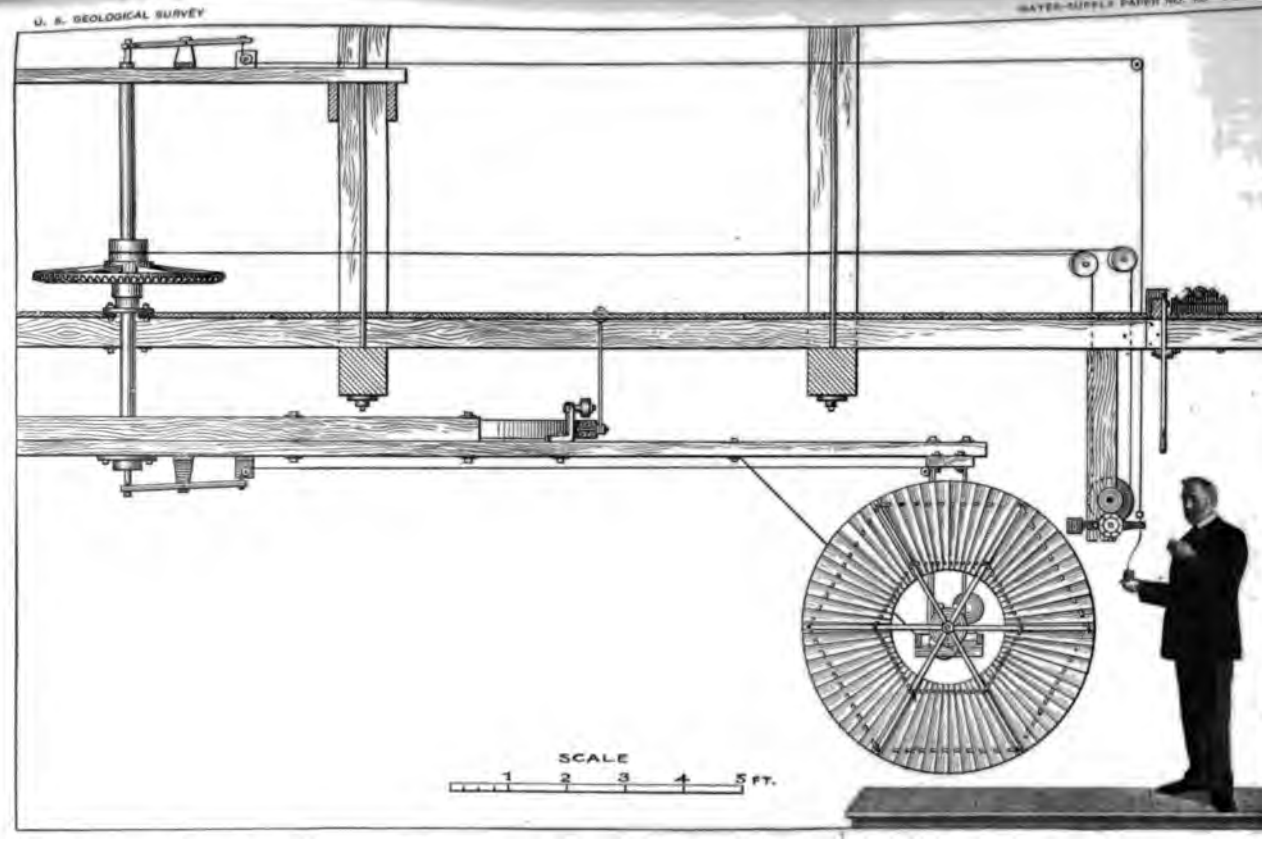
In wheel No. 3 we followed closely Smeaton's angles of weather, but did not obtain so good results as with greater angles. In fact, we were not able to obtain the best results with weather angles so small as Smeaton's in any of our wind wheels. Nor did our sail speeds, as compared with wind velocity, nearly approach the speeds obtained by Smeaton. Even our unloaded wheels did not show the sail speed attained by the best of Smeaton's wheels when loaded for maximum work.

This difference in the ratio of sail speed to wind velocity constituted the greatest disagreement between his results and ours. Our loads at the maximum of work were smaller as compared with greatest loads, and the speeds of revolution at maximum work as compared with speeds of unloaded wheels were smaller for our wheels than for Smeaton's. The general laws established by Smeaton, as enunciated in his eight "maxims," were substantially confirmed by our experiments. The law of cubes pertaining to maximum products and the law of squares pertaining to greatest loads, or starting forces, were more exactly fulfilled in our experiments than in Smeaton's. The differences in results as regards angles of weather, speeds, and best loads might have been due to differences in the construction of the wheels.

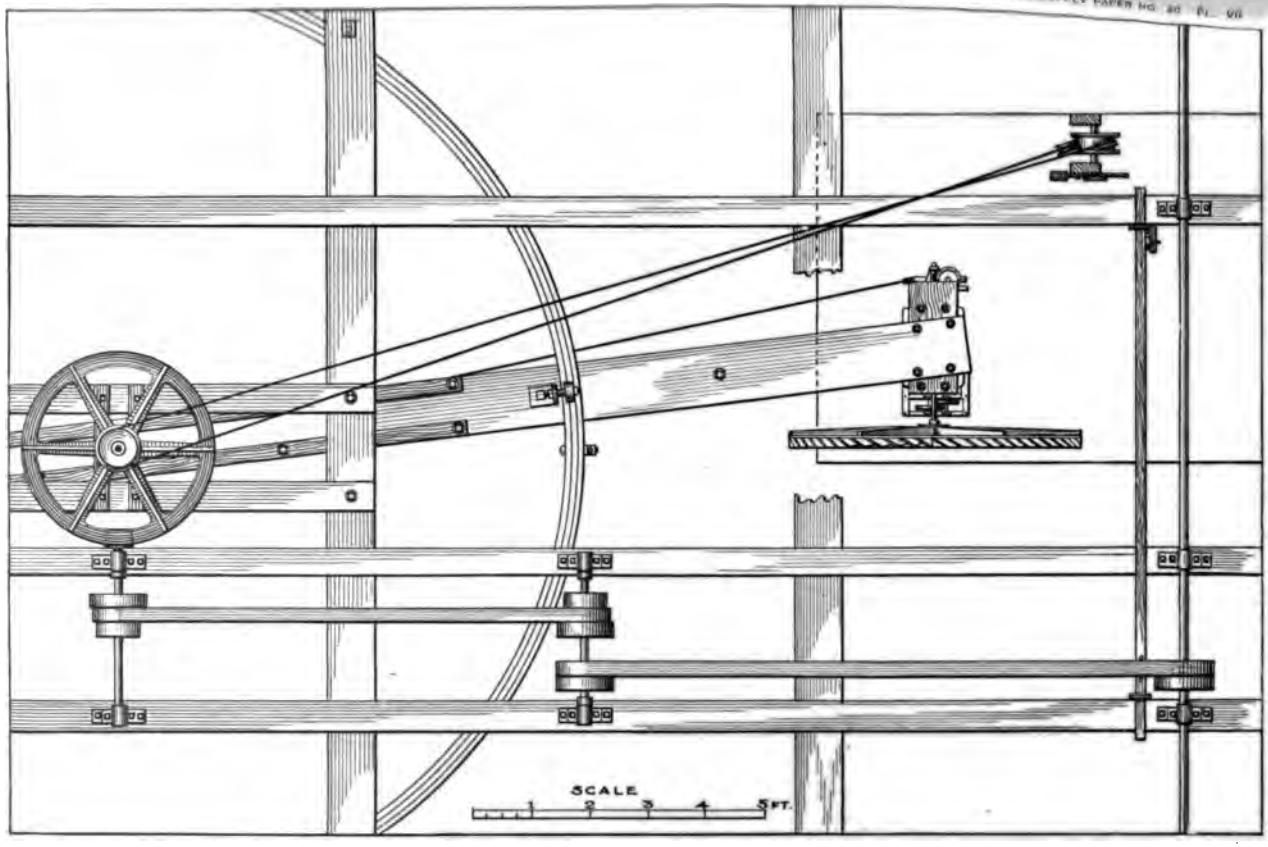
Smeaton constructed his wheels after the manner of windmill construction in his time, and we constructed our wheels mostly after the methods adopted by American manufacturers at the time we experimented. Our methods of experiment were similar, though Smeaton's experiments were conducted on a much smaller scale with smaller wheels, and his appliances generally were crude as compared with our own.

Smeaton allowed about one-eighth of his total applied load as the equivalent additional load due to friction. This friction, as Smeaton determined it, was in reality friction at starting only, and it must have been much in excess of the friction under the rapid revolutions of his wheels, and helped considerably to swell his products.

Still another allowance must be made in comparing Smeaton's efficiencies with ours, on account of the relative shortness of the sweep, which carried his wheels in a circle against still air. The velocity of the centers of his wheels was taken as the equivalent velocity of wind. As a matter of fact, some point beyond the center of the wind wheel, farther away from the axis of the sweep, should have been taken to *represent wind velocity*. The distance from the axis of Smeaton's



ELEVATION OF APPARATUS USED IN WHEEL TESTS.



PLAN OF APPARATUS USED IN WHEEL TESTS.

ep to the extremity of his sail as it revolved was, when greatest, rly double its least distance, and the energy of the wind acting on tip of his sail was more than seven times as great at the maximum at the minimum distance. To compute accurately the total wind rgy intercepted by the wheel, after taking into account the various ocities of impingement, would be an interesting problem in integral culus.

The differential expression for work is easily found, but the integra- n is not so simple. An approximate calculation shows that the wind rgy intercepted by Smeaton's wheel was more than 7 per cent in cess of what it would have been had all points of the wheel met the with the same velocity as at the center. The difference is not so ge as one would expect from the great difference in energy at extreme sitions. We doubt whether anything should be deducted from neaton's products on this account, for only a very limited portion of e sails could have a proper relative velocity with respect to the wind. ur wheels were, of course, similarly affected, although in considera- y less degree.

Weisbach's theoretical formulæ for wind action, if correct, would st a cloud over Smeaton's results; for he concludes his rather elab- ate mathematical discussion with these words: "According to these periments [Smeaton's] the action of the wind in general upon sail heels is greater than is given either by the theory or by Coulomb's periments." Coulomb's very incomplete experiments seem to have een a great source of consolation to those mathematicians whose eories would not permit the realization of efficiencies so great as meaton obtained.

Our own experience with wind wheels in actual use, in widely differ- nt lines of work, and the testimony of many persons using them, onvince us that the efficiencies we obtained and which Smeaton btained in artificial wind are not so great as those commonly obtained ow in natural wind. We consider our experiments valuable princi- ally on account of the comparisons made and the insight they afford nto the causes of waste. Through the courtesy of Mr. H. N. Wade, anager of the United States Wind Engine and Pump Company, of Batavia, Illinois, we are enabled to reproduce the original records of ur experiments on wind wheels as they were made in 1883.

APPARATUS.

Our experiments were conducted in a room about 36 feet wide, 48 eet long, and 19 feet high from floor to roof trusses.

Pls. VI and VII represent, in elevation and plan, as a whole, the pparatus that was used and show portions of the roof trusses under- eath which the sweep was suspended. The rear end of the sweep, ot shown, carried a counterweight, which caused the whole apparatus o balance on the central shaft. Two rollers carried by the sweep on

opposite sides of the suspending shaft ran lightly over a large circular track and prevented vertical oscillations. Pls. VIII and IX present enlarged views, showing the details of the dynamometer and its method of application.

These four plates and the illustrations of wind wheels Nos. 3, 6, 19, 24, 29, 39, 40, 44, 60, and 61 were recently prepared from data in our possession. The illustrations of wheels Nos. 2, 35, and 48 are reproductions of the drawings made for the original records, as are also all the other illustrative figures which are shown in connection with the original records. In computing the figures denoting efficiencies, shown in connection with the illustrations of wheels Nos. 3 and 6, axle friction was called 0.15 pound instead of the 0.3 pound given in the tables, as explained under the heading "Corrections for axle friction."

In these two cases, and also in computing the efficiency of wheel No. 2, allowance was made for temperature as recorded and for atmospheric density, according to the records of the United States Signal Service station at Chicago for 2 p. m. on the days of experiments.

The efficiencies shown in connection with all the other illustrations of wind wheels were obtained by simply multiplying the efficiency for No. 2 by the ratios of products given in the various tables where direct comparison with No. 2 is shown.

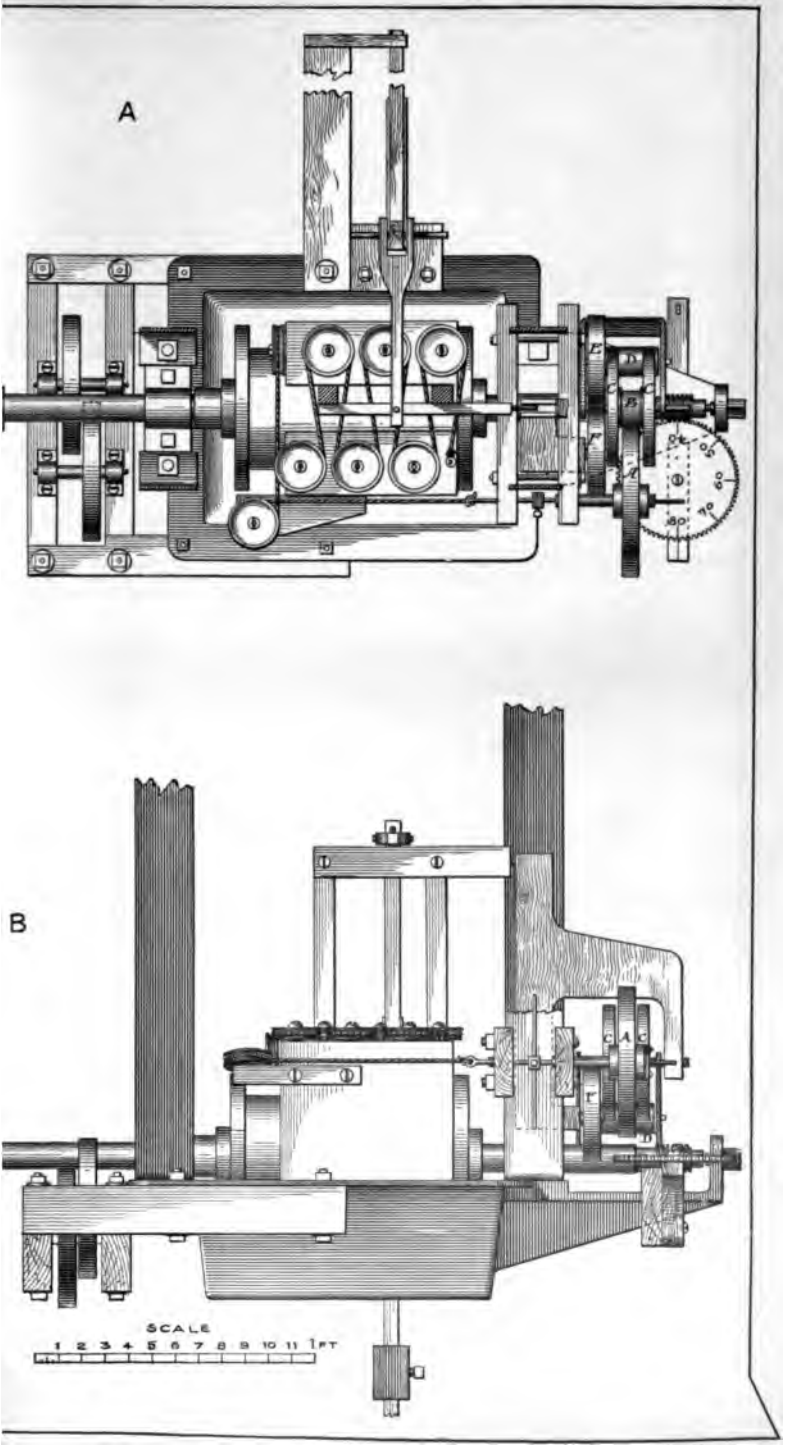
The original records of 1883, immediately following, end on page 72. The comments following page 72, including the tables on pages 74 and 77, have recently been prepared.

ANGLE OF WEATHER.

The term "angle of weather," used in connection with the following tables, means the angle made by the face of the sail with the plane of the wheel.

ARTIFICIAL WIND.

In conducting these experiments with windmills, it was necessary that results to be compared should be obtained in wind of uniform velocity. We therefore made use of artificial wind, obtained by carrying the wind wheels in a circle against still air on the end of a long sweep suspended beneath the roof trusses of a large room used for setting up tanks. The sweep was made to revolve horizontally around its vertical axis by means of gearing, pulleys, and belts connected with a line shaft driven by an 80-horsepower Reynolds–Corliss steam engine, which furnished the motive power for the works of the United States Wind Engine and Pump Company. The distance from axis of sweep to center of wind wheel was 14 feet, so that the velocity of wind against the wind wheel in miles per hour would be indicated very closely by the number of revolutions per minute made by the sweep. To attain absolute accuracy the length of sweep should have been 14.006 feet. Hence it was only necessary to note the number of turns per minute



PLAN AND ELEVATION OF DYNAMOMETER.

de by the sweep in order to know the velocity of wind in miles per ır with sufficient accuracy.

'or counting the number of turns of the sweep a toothed wheel, conning 80 teeth, was used, which was made to revolve at the same rate the sweep. Thus the fractions of a turn could be easily determined within 0.0125, and we made a practice of setting down the fractional ns to within 0.00125, thus securing relative accuracy in determining ; velocity of the wind to within 0.00125 of a mile per hour.

The face of the wind wheel was set at right angles to the direction of e wind. The frame supporting the shaft of the wind wheel was susnded about 3 feet below the sweep by four iron straps, presenting eir thin edges to the wind, so that the action of the wind upon the ıeel should be obstructed as little as possible by the sweep or otherise. The motion of the sweep undoubtedly carried the air of the room ith it to some extent, but not so as to produce a noticeable current.

DYNAMOMETER.

The ordinary Prony friction brake was used, with such modifications ıd additions as circumstances demanded.

The brake consisted of two pine blocks, clamped vertically upon a ass cylinder 5¼ inches in diameter, attached to the shaft of the wind heel. But instead of using two bolts, with thumb nuts for adjusting e brake, one bolt was used as a hinge beneath the cylinder, and the ljustment was made by means of a cord passing across from one ock to the other around small iron sheaves, which turned on common ood screws fastened into the tops of the two blocks, so that the presre of the blocks against the cylinder corresponded to about sixteen nes the tension of the cord.

The adjusting cord was carried back from the brake some distance a direction parallel to the axis of the cylinder to a sheave fixed to e frame supporting the shaft of the wind wheel. From that point e cord was carried by means of sheaves, levers, etc., up through the llow shaft supporting the sweep and finally fastened to one end of a ver. The other end of the lever was hinged and provided with a ding weight, by means of which the tension of the cord could be justed from the station of observation while the sweep was in motion. ıe brake was provided with a horizontal graduated arm carrying a eight, which could be moved in or out to correspond to any desired ıd. Suitable stops were provided, allowing a limited motion of the ake and graduated arm, which could be conveniently watched from e station of observation, just beyond the reach of the sweep. The ake and graduated arm were balanced, as regards the action both of avity and of centrifugal force, before applying the loads, which were lanced by the friction of the brake. The thrust of the rear end of ıe shaft of the *wind wheel* was sustained by a steel point.

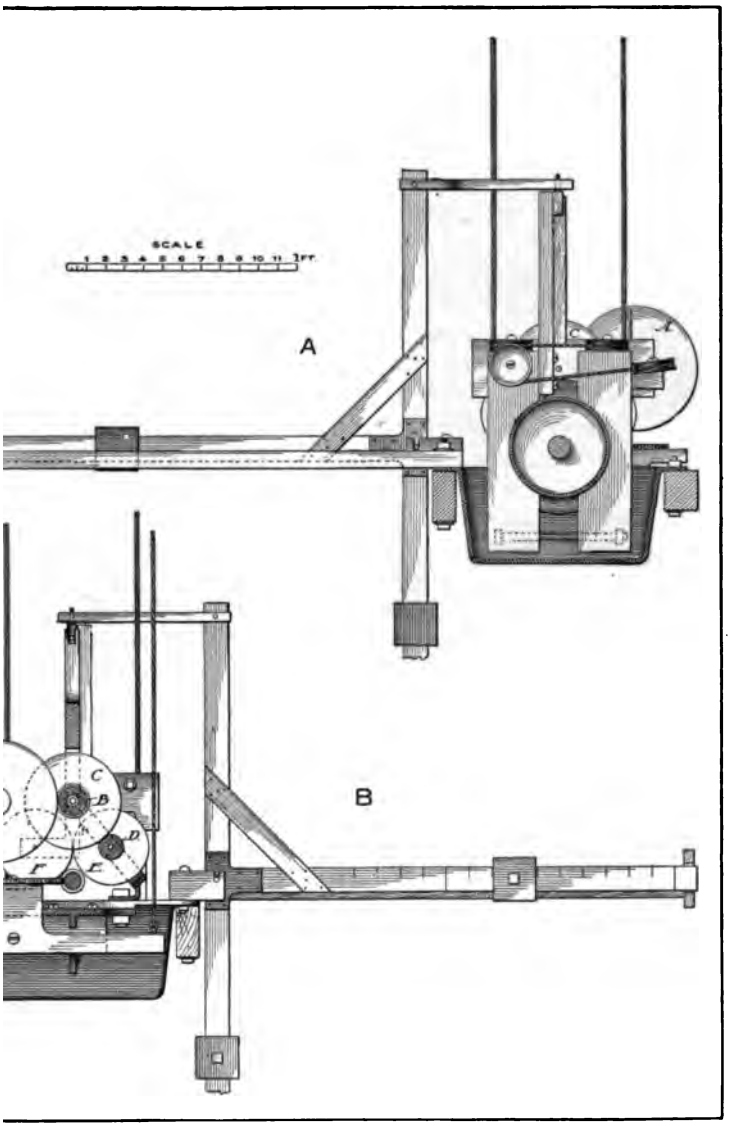
METHOD OF EXPERIMENT.

For noting the number of turns of the wind wheel, the rear end of the shaft was provided with a worm, with which a toothed wheel could be thrown in or out of contact at the pleasure of the operator from the station of observation. Each turn of the wind wheel would cause the toothed wheel to make 0.0125 of a revolution, as there were 80 teeth in the wheel, and the fractional turns could easily be estimated to within 0.1 of a revolution of the wind wheel. The weight on the graduated arm was always expressed in pounds and decimal parts at 1 foot from the center of shaft of the wind wheel. During use the brake was freely lubricated with machine oil.

For a trial load the weight on the graduated arm was set at some number, say 1 pound. Then the counter which was to record the speed of the wind wheel was set at 0, as was also the counter which was to record the velocity of the wind. The sweep having been set in motion, the speed of the wind wheel was checked by the brake, which was adjusted from the station of the observer by regulating the tension of the adjusting cord until the weight on the graduated arm was just lifted by the friction, and no more. As soon as the friction and weight were well balanced the counters which were to record the speed of the wheel and velocity of wind were thrown in by a single movement of a lever, and at the expiration of one minute they were both instantaneously thrown out. Then the sweep was stopped at the station of observation, the number of revolutions of the wind wheel recorded, and also the velocity of the wind.

To the load applied was added the friction of the journals, also expressed in pounds at 1 foot from the center of the shaft. This sum made the total load, which was multiplied by the number of turns of the wind wheel.

Then a second product was obtained in precisely the same manner with a greater load applied; and if greater than the first, a still greater load was applied and the corresponding product obtained. A succession of products were thus obtained corresponding to a succession of loads gradually increased until the products began to decrease on account of the diminishing speed of the wheel. The greatest product corresponded to the best load for the wheel, and the actual work performed by the wheel in each case could be obtained by multiplying the product by 6.2832 feet, equal to the circumference of the circle whose radius is 1 foot. In obtaining these products measurements were taken several times for each load and the total load was multiplied by the average turns of the wheel per minute. The average wind was also taken for record. The velocity of wind was supposed to be nearly uniform, but varied slightly with the speed of the engine, and the variation was sufficient to affect results considerably in some cases; hence the necessity of recording the velocity of the wind in each case. In comparing



END VIEWS OF DYNAMOMETER.

results some allowance also had to be made for differences in velocities of wind.

In testing the speed of the wheel unloaded the brake was removed, making the load nothing but the journal friction.

STARTING FORCES.

The determination of starting forces was attended with some difficulty. In the first place, it became necessary to define what should be considered the starting force of a wind wheel—whether it should be considered the greatest load under which the wheel would turn, or the greatest load which the wheel would balance without turning. It was also necessary to decide what motion of the wheel should be defined as turning. The wheel with a certain load would sometimes perform one revolution in ten or twenty minutes, if that could be called turning; between that and proper continuous turning there were many gradations. It was impracticable to decide when the wheel reached the lowest limit of speed which could properly be called turning. Therefore we have considered the starting force as the greatest load the wheel would balance without turning. Our method of determining starting forces was to clamp the brake so tightly on the cylinder that the wheel could not turn, but could lift the load applied such small distance as was allowed by the stops which limited the motion of the brake. The sweep was then set in motion and timed for one minute, during which time the load on the graduated arm was watched, its alternating upward and downward movements were noted, and judgment was exercised as to the balance between the load applied and the starting force of the wind wheel. The average velocity of wind during these tests was always recorded after March 13, 1883, but not always before that date.

In lifting the load the starting friction of the journals was also overcome by the wind wheel, but when the weight descended the load overcame the starting friction of journals in addition to the starting force of wind, and as the load almost constantly moved up and down when well balanced, nothing should be added to loads applied for friction in case of starting forces. Yet the friction has been added in the tables, the same as to other loads applied, in order to accommodate the statements at the head of columns for loads. On account of the starting friction being great and uncertain, and for other reasons, the starting forces recorded prior to March 13, 1883, are not so much to be relied upon as those recorded after that date; nor were the later tests for starting forces nearly so accurate as those for products.

JOURNAL FRICTION.

For journal friction previous to March 13, 1883, see description in connection with wind wheel No. 11, page 38. After the date above given *the journal friction was greatly* reduced by the use of large anti-

friction wheels in place of the front box, just behind the wind wheel and in front of cylinder, where the weight of the wind wheel and cylinder was mostly supported. In place of the rear box a small steel pin was inserted in the end of the shaft and turned in a brass box. The end thrust of the shaft, due to wind pressure and tension of the adjusting cord, was sustained by a steel point, as before.

As thus improved, one-half an ounce hung to a thread wound around the outside of wheel No. 37 overcame friction at very slow motion, and was almost if not quite equal to starting friction. The brake was removed during this test.

Let x = weight 12 inches from axis required to overcome friction.
30 inches = distance from axis to thread.
$\frac{1}{2}$ ounce = $\frac{1}{32}$ pound.

$$12\,x = \tfrac{30}{32}$$

$$x = 0.0785 = \text{friction at very slow motion or at starting.}$$

With brake and wind wheel removed 26.50 ounces hung by a thread wound around the forward end of shaft 0.7969 inch in diameter overcame friction at very slow motion and was almost equal to starting friction, and 24.25 ounces hung in the same way overcame friction at 50 turns per minute, and would not overcome friction when speed was much less. 0.8125 inch = diameter outside of thread, 0.4024 inch = radius to center of thread.

Friction at 50 turns per minute is to starting friction as 24.25 is to 26.50. Hence, $0.0785 \times \frac{24.25}{26.50} = 0.0714$ pound = journal friction at 50 turns per minute, with wind wheel on and brake removed. The weight of the brake would, of course, add a little to the friction, which we have uniformly called 0.1 in the tables, as explained on page 28, under the table.

The 0.3-pound friction added to loads applied prior to March 13, 1883, was friction at very slow motion and was undoubtedly much in excess of friction at speed of wheels corresponding to maximum products, as is evinced by the fact that notwithstanding the great reduction of starting friction the tables do not show for the same applied loads, or even unloaded, very much increase of speed of wheels after March 13, 1883, over speeds indicated previous to that date. If 0.1 pound had been added to applied loads previous to March 13, 1883, the products at maximum and for higher speeds of wheels would probably have been more nearly correct. It was this uncertainty and ignorance regarding friction at different speeds that led us to reduce as much as possible this source of error. However, the error thus arising does not greatly affect comparative values of products recorded prior to March 13, 1883, although products affected by allowance of 0.3 pound for friction should not, unless corrected, be compared with products recorded after above date.

COMPARATIVE STANDARD WHEEL.

ı the earlier stages of these experiments we acted on the supposi- that only such variations in the state of the atmosphere as were :ated by the barometer and thermometer should affect the work ormed by a wheel tested at different times in wind of the same city. But later we were convinced that the readings of the barom- and thermometer would not always account for variations in lucts given by the same wheel on different days. We were also rinced of the uselessness of speculating as to, or trying to under- d and forestall, all the causes that might combine to affect results. refore we adopted the practice of comparing the maximum product ıch wheel tested with the maximum product of wheel No. 2, obtained ose proximity as to time and under similar conditions in all respects. ı could be done with a good deal of certainty, and no further trouble experienced on account of contradictory results. After the best s had been ascertained by trial, comparative tests were made by ıg first one wheel and then the other in succession until a consid- le number of measurements had been taken. Then as many meas- ıents as possible, giving the same average velocity of wind for both els, were selected from each set of experiments.

this way the product obtained was usually the average of 8 to 10 surements of each wheel made under the same conditions of time, l, weather, and whatever else might affect the results. The prod- of No. 2 was thus used as a standard of measure, and the ratio of .ucts indicated the relative value of the wheel compared.

ɔr further particulars consult tables of experiments of later date ı March 13, 1883.

Results of experiments performed in March and May, 1883, with wheel No. 2.

Velocity of wind per hour.	Load applied + 0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Thermometer.	Date.
Miles.	*Pounds.*			°*F.*	1883.
6.437	0.1	43.78	.....	62	May 14
6.371	1.1	23.90	26.290	54	May 14
6.381	2.4	0	.....	62	May 14
8.417	.1	59.05	.....	53	Mar. 14
8.403	1.9	32.875	62.463	50	Mar. 14
8.405	4.2	0	.....	53	Mar. 14
10.898	.1	77.85	.....	46	May 21
11.041	3.3	41.06	135.498	60	May 17
10.976	6.9	0	.....	46	May 21

revious to making the above experiments, March 13, 1883, the dyna- ɜeter was improved by the addition of an automatic brake adjuster

and antifriction wheels, which reduced starting friction from abc to about 0.08 pound, and made journal friction at 50 turns per r about 0.071 pound. In the table friction was called 0.1 pound fc venience, as the absolute friction in each case could not easily tained, and the error does not materially affect comparative r (See pages 25 and 26 for "Journal friction.")

Average products of wheel No. 2, which was used as a stand comparison, were obtained by selecting from numerous tests s numbers—generally 8 to 10—which corresponded to wind of th average velocity as wind for wheels compared in each case; the ence of averages in no case exceeded 0.001 mile per hour in the ta page 47.

Measurements for maximum products of two wheels compare always taken as far as possible under the same conditions of win perature, barometer, weather, time, etc.

RESULTS OF ORIGINAL TESTS.

TABLES AND DESCRIPTIONS OF WIND WHEELS.

WHEEL NO. 1.—*Results of experiments.*

Velocity of wind per hour.	Load applied +0.3 for friction.	Turns of wheel per minute.	Product.
Miles.	*Pounds.*		
8.427	0.3	59.08	17.724
8.431	1.2	42.60	51.120
8.385	1.4	39.90	55.860
8.412	1.6	35.69	57.104
8.452	1.8	31.69	57.042
8.432	2.0	28.02	56.040
.....	2.85	0	0

Experiments made June 1 and 2, 1882.
Load expressed in pounds applied 1 foot from center of wheel.
Circumference of circle whose radius is 1 foot, 6.2832 feet.
Work per minute at the maximum, $57.104 \times 6.283 = 358.784$ foot-pounds
Diameter of wheel, 5 feet.
Number of slats or sails, 30.
Length of slats, 18 inches.
Width of slat at outer end, 3.56 inches.
Width of slat at inner end, 1.344 inches.
Total area of slats or sails, 9.19 square feet.
Angle of weather, 35 degrees.
Portion of available annular space filled by sails, 0.5.
The spaces between are equal to the spaces occupied by the slats. This v the same as No. 2 with one-half the slats omitted.
Slats, plain surfaces, made of white pine three-sixteenths of an inch thic forward edges trimmed to an edge.

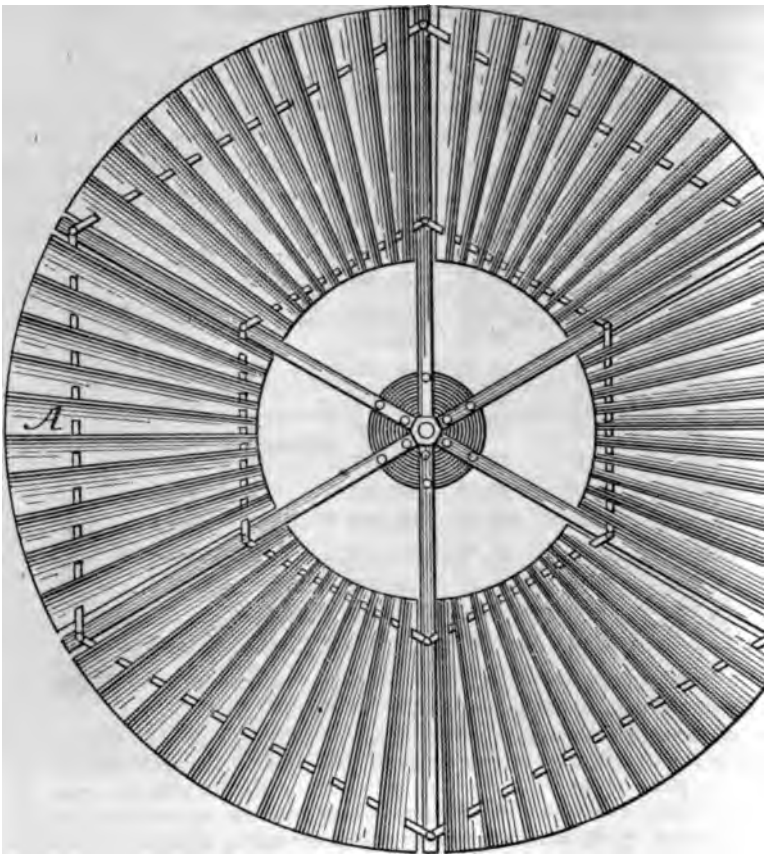
WHEEL NO. 2.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			° *F.*	1882.
8.411	0.3	57.95	17.385	69	.....
8.381	0.9	49.00	44.100	.....	.....
8.451	1.05	47.00	49.350	.....	.....
8.435	1.3	43.08	56.004	77	June 7
8.445	1.5	40.80	61.200	73	June 7
8.412	1.7	37.30	63.410	70	June 7
8.460	1.8	35.85	64.530	78	.....
8.436	1.9	34.89	66.291	72	June 8
8.429	2.0	33.43	66.860	78	.....
8.443	2.1	31.33	65.793	69	June 8
8.421	2.3	27.60	63.480	73	June 8
8.446	2.5	24.48	61.200	75	June 8
8.416	2.7	21.70	58.590	76	June 8
8.398	2.9	20.00	58.000	.....	.....
8.338	3.1	15.80	48.980	.....	.....
.....	3.81	0	0	.....	June 9

tal area of slat sails 18.380 square feet.

gle of weather, 35 degrees.

e same as No. 1, with double the number of slats—that is, the projections of the upon the plane of the wheel equaled the entire available annular area. An as shown in fig. 8 at each of the six arms constituted the space not available. s held in place by 12 straight notched pieces extending from arm to arm the e as in No. 1.



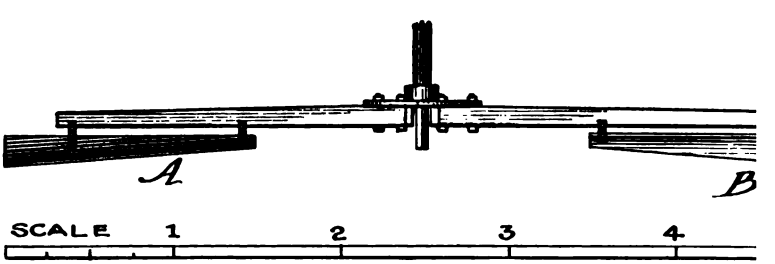


FIG. 7.—Elevation and section of wheel No. 2. (Sail area, 18.380 square feet; efficiency,

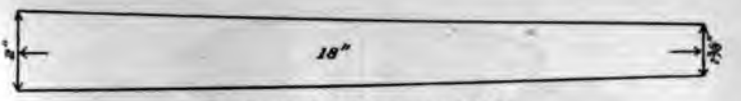


FIG. 8.—Outline of area not available.

WHEEL NO. 3.—*Results of experiments.*

Velocity of wind per hour.	Loads applied+0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			° *F.*	1882.
8.476	0.3	90.48	27.144	68	June 22
8.435	.95	69.40	65.930	68	June 22
8.441	1.05	64.90	68.145	68	June 21
8.428	1.1	62.15	68.365	88	June 22
8.447	1.2	59.65	71.580	69	June 19
8.436	1.3	55.36	71.968	83	June 22
8.402	1.4	51.10	71.540	69	June 21
8.423	1.5	47.48	71.220	74	June 19 June 22
8.441	1.7	41.00	69.700	71	June 22
8.404	1.9	32.00	60.800	71	June 22
.....	2.4	0	0	.....	June 22

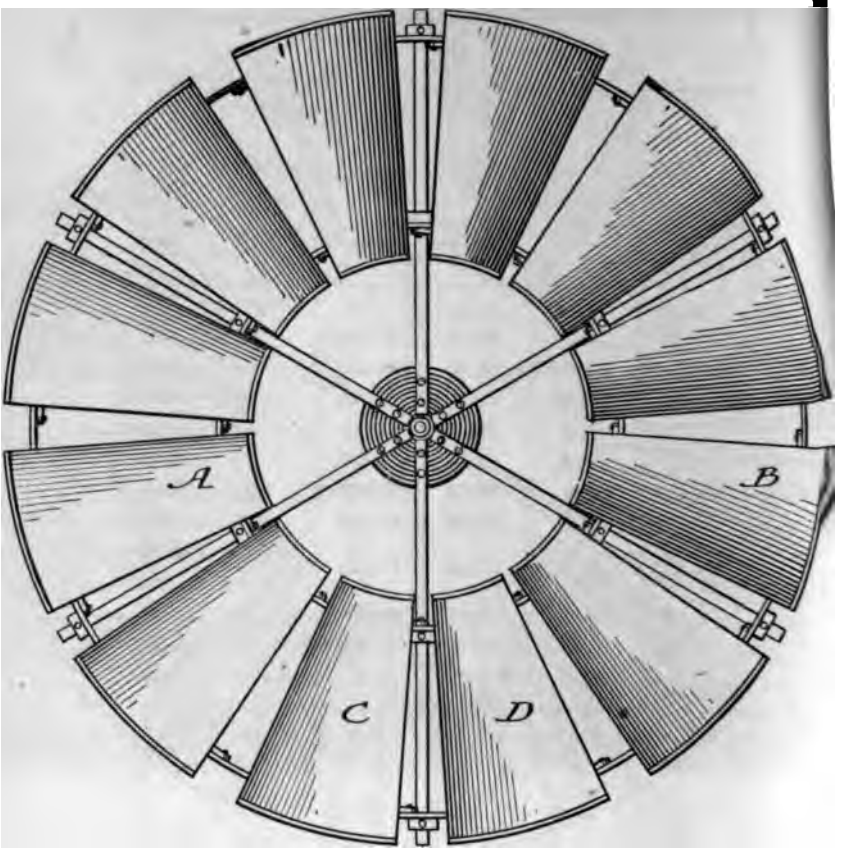


FIG. 9.—Elevation and section of wheel No. 3. (Sail area, 13.590 square feet; efficiency, 0.161.)

Diameter of wheel, 5 feet.

Number of sails, 12.

Sails of lineal dimensions as shown in figs. 10 and 11, made of white-pine board three-sixteenths of an inch thick and fastened by hinges near each end to 12 pin bands bent to arcs of circle and secured to the 6 arms.

Total sail surface, 13.59 square feet.

Sails concave toward the wind.

FIG. 10.—Dimensions of sails of wheel No. 3. 18 inches is the extreme length of sail.

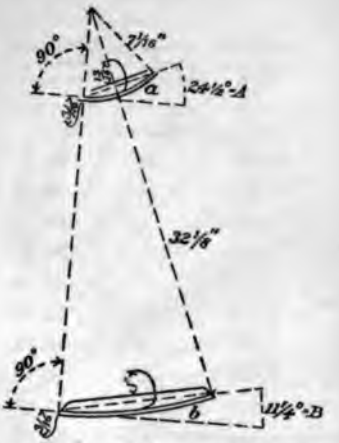


FIG. 11.—Angles of weather of sails of wheel No. 3. *A*. Angle of weather at inner ends of sails; *B*, Angle of weather at outer ends of sails.

WHEEL NO. 4.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			° *F.*	1882.
8.354	0.3	89.43	26.829	76	June 29
8.372	0.9	74.54	67.086	76	June 29
8.375	1.1	70.03	77.033	76	June 29
8.324	1.3	62.75	81.575	68	June 29
8.461	1.5	60.72	91.080	73	June 28
8.467	1.7	55.93	95.081	73	June 28
8.465	1.9	48.75	92.625	73	June 28
8.366	2.1	42.78	89.838	68	June 29
8.386	2.3	38.50	88.550	68	June 29
8.369	2.5	29.13	72.825	68	June 29
.....	3.3	0	0	.....	June 29

ıeel the same as No. 3, except that the inclination of the sails was changed, makhe angle of weather 20 degrees at outer ends of sails and 30 degrees at inner ends ils.

WHEEL NO. 5.—*Results of experiments.* (*a*)

Velocity of wind per hour.	Loads applied +0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			°*F.*	1882.
8.444	0.3	86.78	26.034	74	July 12
8.499	0.9	73.56	66.204	74	July 12
8.479	1.1	68.72	75.592	74	July 12
8.493	1.3	64.18	83.434	71	July 5
8.419	1.5	57.98	86.970	71	July 5
8.422	1.7	53.86	91.562	71	July 5
8.468	1.9	50.83	96.577	71	July 5
8.465	2.1	46.10	96.810	74	July 5
8.456	2.3	41.60	95.680	74	July 5
8.414	2.5	33.86	84.650	63	July 6
8.424	2.7	27.70	74.790	63	July 12
8.440	2.9	19.62	56.898	63	July 12
.....	3.7	0	0	.....	July 5
8.491	1.9	51.88	98.572	71	.....

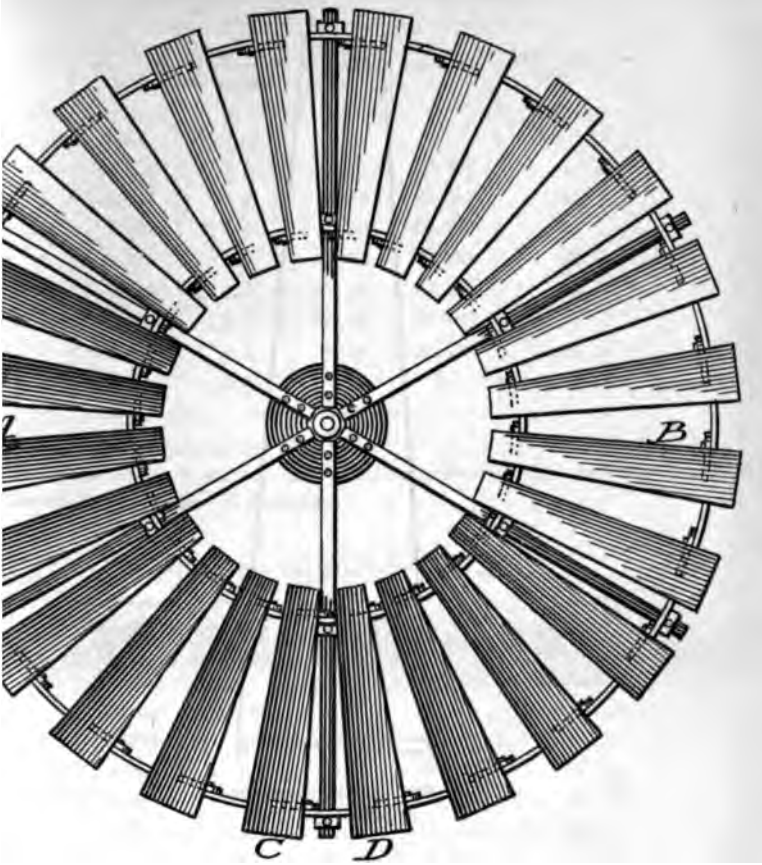
a See wheel No. 44, p. 57.

Wheel the same as No. 3, except that the inclination of the sails was chang making the angle of weather 22.5 degrees at outer ends of sails and 32.5 deg at inner ends of sails.

WHEEL NO. 6.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			°*F.*	1882.
8.373	0.3	69.03	20.709	32	Dec. 9
8.355	1.5	47.30	70.950	34	Dec. 9
8.348	1.7	44.00	74.800	34	Dec. 9
8.405	1.8	42.10	75.780	35	Dec. 9
8.317	1.9	38.28	72.732	35	Dec. 9
a 8.368	1.3	50.83	66.079	38	Dec. 9
.....	3.4	0	0	33	Dec. 8

a When reversed, so as to present its back side to the wind.



Elevation and section of wheel No. 6. (Sail area, 10.688 square feet; efficiency, 0.163.)

er of wheel, 5 feet.
ıd bands the same as in wheel No. 3.
· of sails, 24. Sails of lineal dimensions as in fig. 13; made of white-pine
ree-sixteenths of an inch thick, and hinged to the bands as in No. 3.
ain, with forward edges trimmed to an edge.
ıil surface, 10.688 square feet.
f weather, 30 degrees.

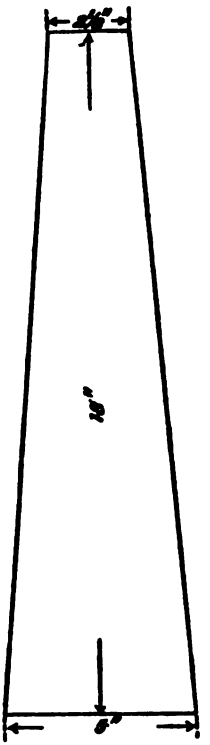


FIG. 13.—Dimensions of sails of wheel No. 6.

WHEEL NO. 7.—*Results of experiments.*

Velocity of wind per hour.	Loads applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			°*F.*	1882.
8.361	0.3	73.60	22.080	42	Dec. 12
8.380	1.3	52.50	68.250	39	Dec. 12
8.416	1.5	47.40	71.100	40	Dec. 11
8.359	1.7	40.74	69.258	40	Dec. 12
.....	2.8	0	0	42	Dec. 12

Wheel the same as No. 6, except that the inclination of the sails was ch making the angle of weather 25 degrees.

WHEEL No. 8.—*Results of experiments.*

Velocity of wind per hour.	Loads applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			°*F.*
8.333	0.3	62.67	18.801	.....
8.344	1.3	46.83	60.879	48
8.403	1.5	44.23	66.345	48
8.375	1.7	40.23	68.391	39
8.366	1.9	36.84	69.996	41
8.368	2.1	32.80	68.880	47
8.354	2.3	27.70	63.710	47
.....	3.55	0	0	.....

nts made December 13, 1882.
e same as No 6, except that the inclination of the sails was changed,
angle of weather 35 degrees.

WHEEL No. 9—*Results of experiments.*

Velocity of wind per hour.	Loads applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			°*F.*
8.374	0.3	66.40	19.920	43
8.361	1.3	49.73	64.649	38
8.375	1.5	47.15	70.725	39
8.350	1.7	42.30	71.910	40
8.364	1.9	38.20	72.580	40
8.344	2.1	34.58	72.618	42
8.323	2.3	30.70	70.610	42
.....	3.4	0	0	.....

nts made December 14, 1882.
e same as No. 6, except that the inclination of the sails was changed,
angle of weather 32.5 degrees.

WHEEL No. 10.—*Results of experiments.*

Velocity of wind per hour.	Loads applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			°*F.*	1882.
8.347	0.3	72.23	21.690	34	Dec. 15
8.336	1.1	54.77	60.247	45	Dec. 14
8.329	1.3	53.30	69.290	45	Dec. 14
8.345	1.5	48.70	73.050	33	Dec. 15
8.358	1.7	43.78	74.426	34	.....
8.338	1.9	39.33	74.727	35	.....
8.351	2.1	35.06	73.626	39	.....
.....	3.25	0	0	.....	.....

Wheel the same as No. 6, except that the inclination of the sails was ch making the angle of weather 27.5 degrees.

WHEEL No. 11.—*Results of experiments.*

Velocity of wind per hour.	Loads applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			°*F.*
8.415	0.3	77.60	23.280	41
8.353	1.3	51.64	67.132	39
8.391	1.5	45.46	68.190	40
8.401	1.7	39.52	67.184	40
8.410	1.9	*a* 33.14	62.966	41
.....	2.1	(*b*)	.....	.....
.....	2.3	0	0	.....

a Started slowly and with difficulty, though well balanced. Once needed help to start.
b Ran slowly two or three minutes and stopped.

Experiments made December 18, 1882.

Wheel the same as No. 6, except that the inclination of the sails was ch making the angle of weather 20 degrees.

A journal friction test made with wheel No. 11 by hanging a 1¾-ounce weig a thread wound around the outside of the wheel showed that this weight d overcome starting friction, but was sufficient to accelerate motion after the was started at moderate speed.

Let x = journal friction, at 1 foot (12 inches) from axis.

30 inches = distance of 1¾-ounce weight from axis.

$1.75 \times 30 = 12\,x$.

$x = 4.375$ ounces = 0.2734 pound = friction of journals at moderate speed.

The journal friction diminishes somewhat with the increase of speed, alt authorities give the same coefficient of friction at all speeds. Some variat journal friction is also due to difference in weights of wheels and in lubric and as the above test was made with the brake removed, we have conside approximately correct to call journal friction = 0.3 pound in all cases. See ever, later conclusions.

WHEEL NO. 12.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.405	0.3	75.60	22.680	44
8.385	.95	56.18	53.371	40
8.376	1.1	50.78	55.858	40
8.341	1.3	*a* 43.88	57.044	41
8.376	1.5	*b* 35.42	53.130	42
.....	1.6	(*c*)	.....	.....
.....	1.7	0	0	.....

a Started slowly.

b Needed assistance to start in every instance, though without assistance it would barely move, at the rate, perhaps, of one turn in ten minutes.

c Ran three minutes and stopped, after having been started at moderate speed.

Experiments made December 19, 1882.

Wheel the same as No. 6, except that the inclination of the sails was changed, making the angle of weather 15 degrees.

WHEEL NO. 13.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.	Date.
Miles.	*Pounds.*			° *F.*	1882.
8.369	0.3	56.33	16.899	45	Dec. 20
8.377	1.7	37.13	63.121	45	Dec. 19
8.353	1.9	34.10	64.790	43	Dec. 20
8.352	2.1	31.30	65.730	41	Dec. 20
8.407	2.3	28.70	66.010	42	Dec. 20
8.375	2.5	25.38	63.450	42	Dec. 20
8.328	2.7	20.63	55.701	44	Dec. 20
8.351	2.9	17.25	50.025	46	Dec. 20
.....	3.1	(*a*)	.....	.....	Dec. 20
.....	3.2	(*b*)	.....	.....	Dec. 20
.....	3.7	0	0	.....	Dec. 20

a Ran very slowly after starting, and stopped several times.

b Stopped and started occasionally.

Wheel the same as No. 6, except that the inclination of the sails was changed, making the angle of weather 40 degrees.

WHEEL NO. 14.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.327	0.3	48.60	14.580	47
8.335	1.5	33.60	50.400	46
8.375	1.7	32.68	55.556	46
8.393	1.9	29.88	56.772	47
8.336	2.1	26.95	56.595	46
8.364	2.3	24.72	56.856	46
8.427	2.5	21.82	54.550	46
8.381	2.7	17.98	48.546	46
8.409	2.9	15.10	43.790	46
8.349	3.1	12.47	38.657	46
.....	3.3	(*a*)	.....	.....
.....	3.8	0	0	.....

a Ran a minute or two, stopped, and afterwards started slightly.

Experiments made December 21, 1882.

Wheel the same as No. 6, except that the inclination of the sails was chang making the angle of weather 45 degrees.

WHEEL NO. 15.—*Results of experiments.*

Velocity of wind per hour.	Load applied +0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.444	0.3	46.84	14.052	30
8.402	1.7	29.36	49.912	28
8.429	1.9	27.40	52.600	30
8.479	2.1	25.30	53.130	31
8.457	2.3	22.92	52.716	33
8.443	2.5	20.17	50.425	32
8.440	2.7	17.71	47.817	33
8.438	3.1	10.20	31.620	35
.....	3.3	(*a*)	.....	30
.....	3.5	(*b*)	.....	30
8.406	3.95	0	0	35

a Turned very slowly and irregularly. *b* Barely moved.

Experiments made January 5 and 6, 1883.

Wheel the same as No. 6, except that the inclination of the sails was char making the angle of weather 47.5 degrees.

WHEEL NO. 16.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.443	0.3	42.40	12.720	32
8.444	1.7	27.75	47.175	32
8.440	1.9	24.92	47.348	30
8.385	2.1	23.27	48.867	30.5
8.447	2.3	20.67	47.514	38
8.419	2.5	18.45	46.125	37
8.505	2.7	16.23	43.821	37
8.426	2.9	*a* 12.33	35.757	37
8.484	3.1	(*b*)	.....	36
.....	3.3	(*c*)	.....	36
8.257	3.55	0	0	35
8.426	3.9	0	0	36

a Almost needed help to start, and once stopped within one minute.
b Stopped after running one minute.
c Stopped after running very slowly a short time.

periments made January 6 and 8, 1883.
ıeel the same as No. 6, except that the inclination of the sails was changed,
ng the angle of weather 50 degrees.

Comparative results with plain sails.

A	B	C	D	E	F	G	H	I
No. wheel.	Angle of weather.	Velocity of wind per hour at maximum.	Greatest load.	Load at maximum.	Turns of wheel per minute at maximum.	Turns of wheel per minute unloaded.	Products at maximum work, E × F.	Total sail surface.
	°	*Miles.*	*Pounds.*	*Pounds.*				*Sq. ft.*
1	35	8.412	2.85	1.6	35.69	59.08	57.104	9.190
2	35	8.429	3.81	2.0	33.43	57.95	66.860	18.380
12	15	8.341	1.7	1.3	43.88	75.60	57.044	10.688
11	20	8.391	2.3	1.5	45.46	77.60	68.190	10.688
7	25	8.416	2.8	1.5	47.40	73.60	71.100	10.688
10	27.5	8.358	3.25	1.7	43.78	72.23	74.426	10.688
6	30	8.348	3.4	1.7	44.00	69.03	74.800	10.688
9	32.5	8.364	3.4	1.9	38.20	66.40	72.580	10.688
8	35	8.366	3.55	1.9	36.84	62.67	69.996	10.688
13	40	8.352	3.7	2.1	31.30	56.33	65.730	10.688
14	45	8.336	3.8	2.1	26.95	48.60	56.595	10.688
15	47.5	8.479	3.95	2.1	25.30	46.84	53.130	10.688
16	*50*	*8.385*	*3.9*	2.1	23.27	42.40	48.867	10.688

No. 2 modeled after Halliday 10-foot mill, and contained 60 sails 18 inches long and 3.56 inches wide at outer end.

No. 1 contained 30 sails 18 inches long and 3.56 inches wide at outer end.

Nos. 6–16 contained 24 sails 18 inches long and 5 inches wide at outer end.

If the separate tables are consulted, it will be found that in making out the comparative table, we have not taken the highest products for wheels Nos. 6, 9, 10, 13, and 14, in column H. As the angle of weather increases from 15 degrees to 47.5 degrees, the loads at maximum should obviously increase or at least should not decrease in any case within those limits. This law would apparently have been violated if we had taken the highest products in the above-mentioned cases. So, in Nos. 6, 9, 10, 13, and 14, we have taken the loads and products immediately preceding those corresponding to maximum. By so doing we made column E appear consistent with the law of increasing loads without materially changing the values of products in column H. For it will be noted that in no case does the product we have set down in column H fall short of the highest we might have taken by so much as unity.

The majority of the slight discrepancies in column H are clearly accounted for by the variation in wind as shown in column C. But in Nos. 9 and 10 there was evidently a slight fault in adjusting the friction of the brake to the load applied. Slight errors of this nature are unavoidable, although they may be reduced to a minimum by the exercise of care and repeated trials. (See page 27.)

In the foregoing experiments seldom less than six measurements were made for the determination of each product and sometimes the unsteadiness of the wind required many more trials for the determination of a product. The variations in wind, though not great, sometimes made a difference of one or two turns of the wheel per minute. The indicator which registered the number of turns of the wheel was not originally intended to indicate fractions of a turn, although the fractional turns were actually set down by estimation to within one-tenth of a turn. But the indicator could not be counted on as absolutely correct to within less than half a turn, so that an error of half a turn too much or too little might sometimes account for a variation of one turn of the wheel. Since, however, the indicator was just as liable to make the fractional error one way as the other, the errors in average results would be probably diminished in proportion to the number of tests from which the averages were deduced.

The indicator consisted of a toothed wheel which was thrown in contact with a worm on the shaft of the wheel, and the teeth were liable to strike the worm so as to accelerate or retard the indicator to the extent of half the pitch of the teeth or less.

An examination of the several tables shows that there are generally three or four consecutive products corresponding to different loads, none of which vary much from the greatest product, and when the two highest products are nearly identical as often happens, it requires but a very slight error in any one of two or three chances for variation to make the highest product correspond to either of two consecutive loads.

WHEEL NO. 17.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F*
8.392	0.3	73.90	22.170	32
8.379	1.3	52.62	68.406	30
8.363	1.5	46.55	69.825	30
8.390	1.7	42.80	72.760	31
8.408	1.9	38.70	73.530	31
8.373	2.1	33.30	69.930	31
8.400	3.05	0	0	32
a 8.391	.3	79.83	23.949	32
a 8.380	1.3	54.15	70.395	32
a 8.374	1.5	47.60	71.400	31
a 8.361	1.7	40.70	69.190	32

a Reversed.

xperiments made January 10 and 11, 1883.

heel the same as No. 6, except that the sails were twisted, making the angle of ther 25 degrees at outer extremities of sails and 35 degrees at inner extremities ails.

he twisting made the sails somewhat convex on the front side and concave on reversed side.

WHEEL NO. 18.—*Results of experiments.* (*a*)

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.430	0.3	75.58	22.674	37
8.370	1.3	53.48	69.524	37
8.362	1.5	50.64	75.960	38
8.386	1.7	48.08	81.736	38
8.397	1.9	43.31	82.289	39
8.393	2.1	39.42	82.782	40
8.357	2.3	33.65	77.395	39
8.368	2.5	*b* 28.31	70.775	40
8.419	2.7	*c* 21.83	58.941	37
8.440	3.4	0	0	38

a See table on p. 47.
b Started slowly.
c Started very slowly—almost needed help.

xperiments made January 13 and 17, 1883.

Vheel contained 24 sails of lineal dimensions as shown in fig. 14, made of white- s boards three-sixteenths inch thick, beveled on forward edges, and hinged to bands the same as in wheel No. 6.

otal area of sail surface, 12.937 square feet.

ngle of weather, 25 degrees.

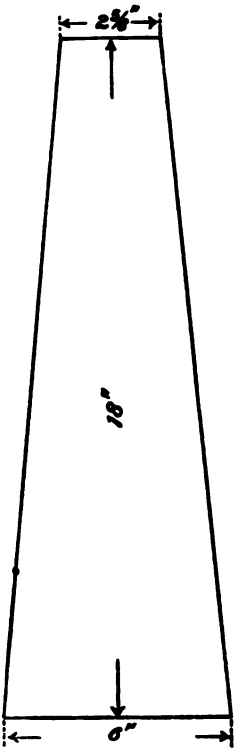
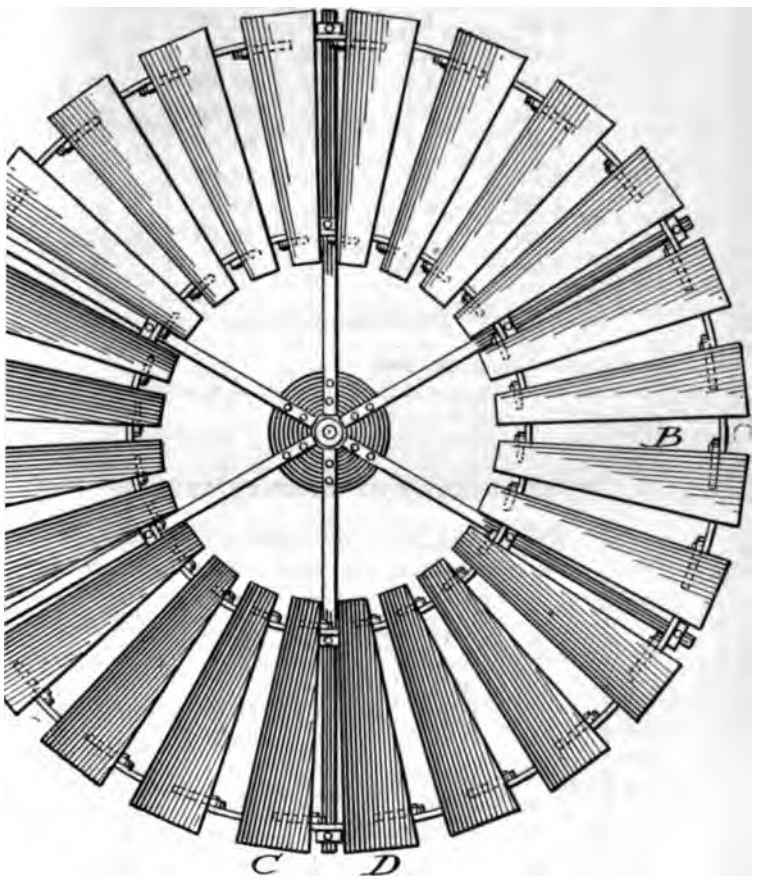


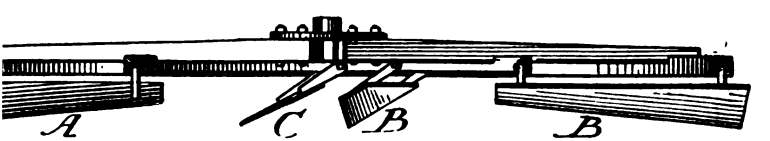
FIG. 14.—Dimensions of sails of wheel No. 18.

WHEEL NO. 19.—*Results of experiments.* (*a*)

Velocity of wind per hour.	Load applied +0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.375	0.3	72.26	21.678	33
8.375	1.3	53.78	69.914	34
8.466	1.5	50.32	75.480	28
8.475	1.7	47.17	80.189	28
8.379	1.9	44.48	84.512	29
8.422	2.1	42.10	88.410	30
8.439	2.3	38.20	87.860	31
8.400	2.5	32.95	82.375	31
8.397	2.7	26.98	72.846	31
8.392	2.9	22.03	63.887	32
8.370	3.1	*b* 16.03	49.693	32
8.329	3.6	0	0	32

a See table on page 47. *b* Needed assistance to start.





Elevation and section of wheel No. 19. (Sail area, 12.937 square feet; efficiency, 0.185).

ıents made January 18, 1883.
the same as No. 18, except that the inclination of the sails was changed,
ıe angle of weather 27.5 degrees.

WHEEL NO. 23.—*Results of experiments.*

Velocity of wind per hour.	Load applied + 0.3 for friction.	Turns of wheel per minute.	Product.	Thermometer.
Miles.	*Pounds.*			° *F.*
8.422	0.3	73.46	22.038	34
8.420	1.3	55.60	72.280	30
8.428	1.5	50.83	76.245	30
8.486	1.7	47.64	80.988	30
8.444	1.9	43.21	82.099	30
8.400	2.1	38.14	80.094	30
8.402	2.3	34.30	78.890	31
8.382	2.5	30.12	75.300	31
8.434	*a* 2.7	24.23	65.421	32
8.399	3.55	0	0	32

a Needed assistance to start.

Experiments made January 25, 1883.

Wheel the same as No. 18, except that the angle of weather was 25 degrees at ou' ends of sails and 30 degrees at inner ends.

COMPARISON OF SEVERAL WHEELS.

WHEELS NOS. 2 AND 50.—*Results of experiments.*

No. of wheel.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Date.
	Miles.	*Pounds.*		1883.
2	11.022	0.1	77.75	July 2
2	11.054	3.3	38.48	July 2
2	10.926	6.8—	0	July 2
50	11.093	0.1	77.45	July 2
50	11.054	3.3	36.36	July 2
50	10.981	6.6	0	July 2

Wheel No. 50 was No. 2 with the center of wheel filled in by a cone whose was 24 inches in diameter and whose slant height was 18 inches. Base of cone re against the front of arms of wheel. Cone made of stiff paper, supported by l wooden frame.

WHEELS NOS. 18 TO 22.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
18	25	8.516	0.1	77.13	.....	.....	.....	Apr. 10
18	25	8.485	1.5	46.20	69.300	58.083—	1.193	Apr. 10
18	25	8.444	3	0	.....	.....	.....	Apr. 10
19	27.5	8.471	0.1	74.94	.....	.....	.....	Apr. 12
19	27.5	8.500	1.9	39.57	75.185	62.719+	1.204	Apr. 11
19	27.5	8.455	3.3	0	.....	.....	.....	Apr. 11
20	30	8.464	0.1	70.78	.....	.....	.....	Apr. 14
20	30	8.486	1.9	39.87	75.753	62.966—	1.203	Apr. 12
20	30	8.475	3.55	0	.....	.....	.....	Apr. 14
21	32.5	8.478	0.1	67.97	.....	.....	.....	Apr. 16
21	32.5	8.463	1.9	38.72	73.568	*a*62.225=	1.182	Apr. 16
21	32.5	8.400	3.7	0	.....	.....	.....	Apr. 16
22	35	8.498	0.1	64.20	.....	.....	.....	Apr. 17
22	35	8.461	2.1	31.96	67.166	58.957—	1.138	Apr. 17
22	35	8.504	3.9	0	.....	.....	.....	Apr. 17

a The sign = indicates that the number affected needs no correction.

These wheels differed from each other only in angles of weather.

WHEELS NOS. 24 TO 28.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
26	25	8.455	0.1	75.38	.....	.....	.....	Apr. 9
26	25	8.434	1.7	43.74	74.358	*a* 62.966+	1.181	Apr. 9
26	25	8.448	3.25	0	.....	.....	.....	Apr, 9
24	27.5	8.428	0.1	72.60	.....	.....	.....	Apr. 7
24	27.5	8.426	1.9	38.87	73.853	*a* 60.762—	1.212	Apr. 6
24	27.5	8.369	3.5	0	.....	.....	.....	Apr. 7
25	30	8.377	0.1	69.00	.....	.....	.....	Apr. 5
25	30	8.450	1.9	38.95	74.005	*b* 61.902—	1.196	Apr. 6
25	30	8.425	3.7	0	.....	.....	.....	Apr. 5
28	32.5	8.437	0.1	67.07	.....	.....	.....	Apr. 4
28	32.5	8.391	2.1	34.49	72.429	*c* 61.142—	1.184	Apr. 4
28	32.5	8.379	3.9	0	.....	.....	.....	Apr. 4
27	35	8.473	0.1	63.65	.....	.....	.....	Apr. 3
27	35	8.401	2.1	33.74	70.854	*d* 61.370+	1.154	Apr. 3
27	35	8.454	4.1	0	.....	.....	.....	Apr. 2

a Wind, 8.432 miles per hour.
b Wind, 8.455 miles per hour.
c Wind, 8.398 miles per hour.
d Wind, 8.395 miles per hour.

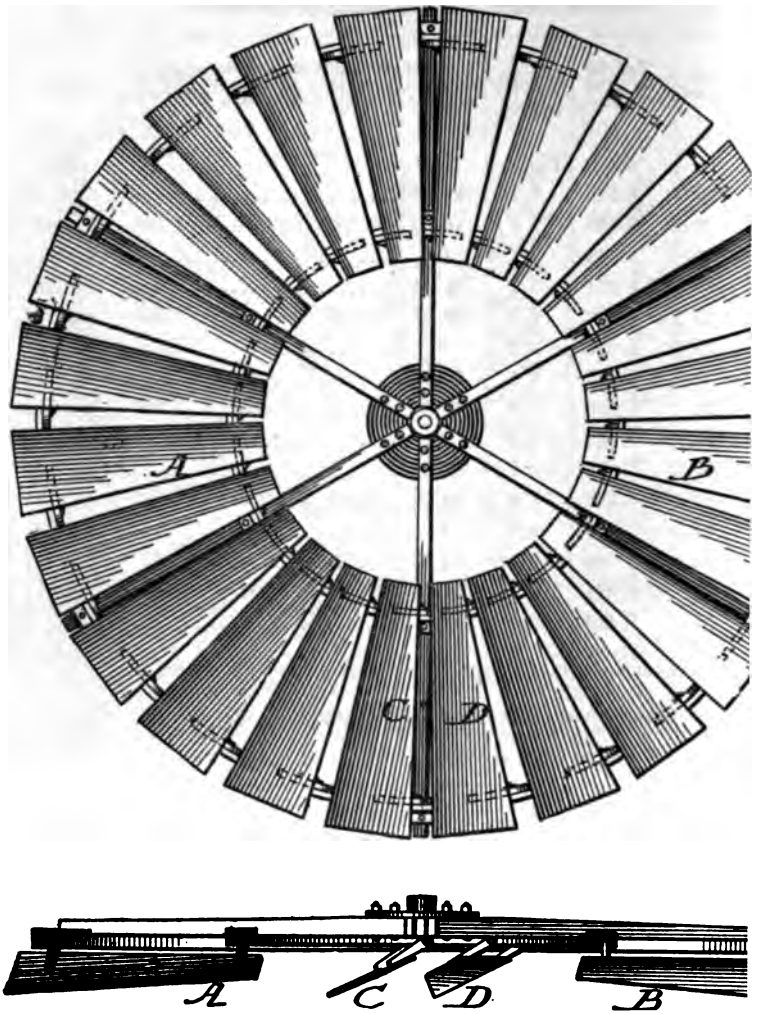


FIG. 16.—Elevation and section of wheel No. 24. (Sail area, 15.000 square feet; efficien

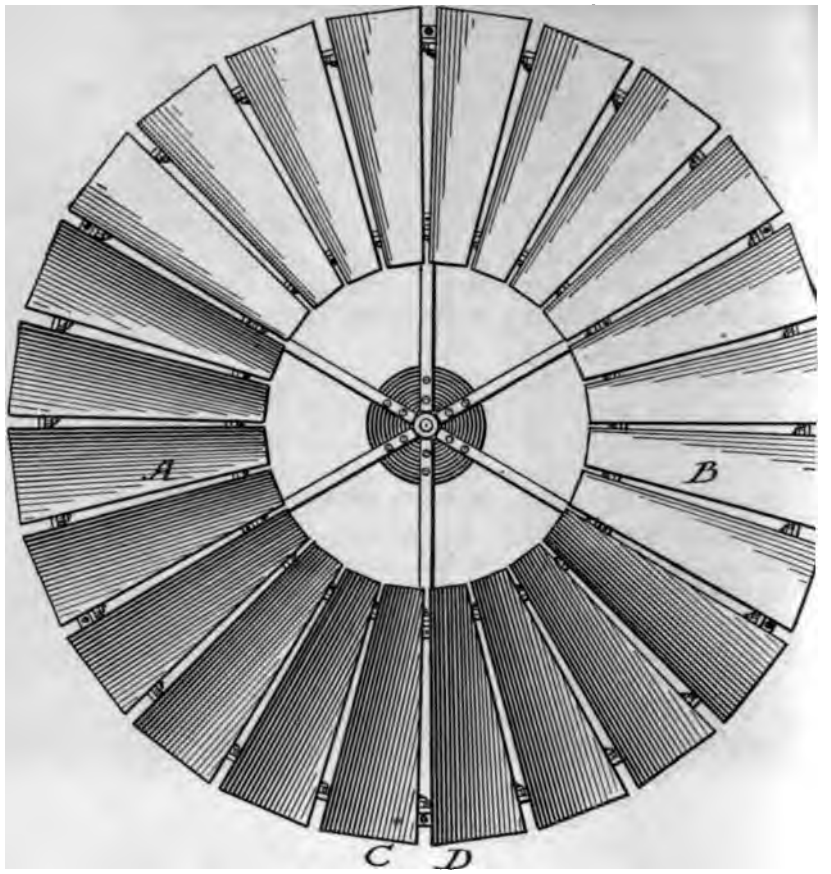
Wheels Nos. 24 to 28 differed from each other only in angles of wea having 24 plain sails 18 inches long, 7 inches wide at outer ends, and 3 in at inner ends. Wheels otherwise the same as No. 6.

Total area of sail surface, 15.000 square feet.

WHEELS NOS. 29 TO 33.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
33	25	8.434	0.1	75.46	.....	.....	.....	Mar. 14
33	25	8.401	1.9	37.80	71.820	*a* 62.463—	1.150	Mar. 14
33	25	8.341	3.5	0	.....	.....	.....	Mar. 14
29	27.5	8.421	0.1	73.10	.....	.....	.....	Mar. 27
29	27.5	8.406	1.9	39.10	74.290	*b* 61.845—	1.201	Mar. 27
29	27.5	8.454	3.7	0	.....	.....	.....	Mar. 27
30	30	8.348	0.1	69.18	.....	.....	.....	Mar. 29
30	30	8.429	2.1	35.15	73.815	*c* 62.478+	1.181	Mar. 28
30	30	8.399	4.0	0	.....	.....	.....	Mar. 28
31	32.5	8.407	0.1	66.73	.....	.....	.....	Mar. 30
31	32.5	8.418	2.1	34.94	73.374	*d* 61.864—	1.186	Mar. 30
31	32.5	8.422	4.1	0	.....	.....	.....	Mar. 30
32	35	8.410	0.1	62.90	.....	.....	.....	Mar. 31
32	35	8.454	2.1	33.43	70.203	*e* 61.408+	1.143	Mar. 31
32	35	8.444	4.3	0	.....	.....	.....	Mar. 31

a Wind, 8.403 miles per hour.
b Wind, 8.410 miles per hour.
c Wind, 8.427 miles per hour.
d Wind, 8.419 miles per hour.
e Wind, 8.446 miles per hour.



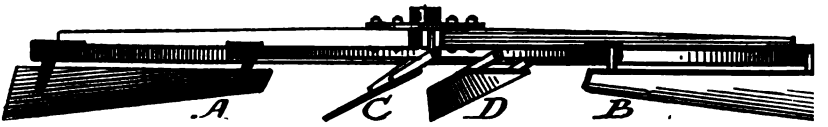


FIG. 17.—Elevation and section of wheel No. 29. (Sail area, 17.016 square feet; efficiency, 0.185.

Wheels Nos. 29 to 33 differed from each other only in angles of weather, each ha ing 24 plain sails 18 inches long, $8\frac{1}{16}$ inches wide at outer ends, and $3\frac{9}{16}$ inches wi at inner ends.

Total area of sail surface, 17.016 square feet.

Wheels otherwise the same as No. 6.

Dynamometer improved by addition of antifriction wheels and automatic bra adjuster, first used March 13, 1883, with wheel No. 33.

WHEELS NOS. 19, 20, AND 34.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
19	27.5	8.471	0.1	74.94	.....	.....	.....	Apr. 12
19	27.5	8.500	1.9	39.57	75.183	*a* 62.719+	1.204	Apr. 11
19	27.5	8.455	3.3	0	.....	.....	.....	Apr. 11
20	30	8.464	0.1	70.78	.....	.....	.....	Apr. 14
20	30	8.486	1.9	39.87	75.753	*b* 62.966—	1.203	Apr. 13
20	30	8.475	3.55	0	.....	.....	.....	Apr. 14
34	25, 35	8.498	0.1	76.47	.....	.....	.....	Apr. 19
34	25, 35	8.502	1.7	43.77	74.409	*c* 61.655+	1.207	Apr. 18
34	25, 35	8.487	3.5	0	.....	.....	.....	Apr. 19

a Wind, 8.499 miles per hour. *b* Wind, 8.487 miles per hour. *c* Wind, 8.501 miles per hour.

Wheels Nos. 19, 20, and 34 differed from No. 18 only in angles of weather.

The sails of No. 34 were twisted, making angles of weather 25 degrees at outer ends of sails and 35 degrees at inner ends.

WHEELS NOS. 35 TO 38.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
38	22.5	8.524	0.1	84.65	.....	.....	.....	Apr. 28
38	22.5	8.488	1.5	52.51	78.765	*a* 63.802=	1.235	Apr. 28
38	22.5	8.521	2.9	0	.....	.....	.....	Apr. 28
36	25	8.525	0.1	80.67	.....	.....	.....	Apr. 26
36	25	8.499	1.7	47.05	79.985	*b* 62.852—	1.273	Apr. 26
36	25	8.413	3.1	0	.....	.....	.....	Apr. 26
35	27.5	8.413	0.1	76.78	.....	.....	.....	Apr. 24
35	27.5	8.424	1.7	46.64	79.288	*c* 63.726—	1.244	Apr. 25
35	27.5	8.445	3.4	0	.....	.....	.....	Apr. 25
37	30	8.459	0.1	73.12	.....	.....	.....	Apr. 27
37	30	8.477	1.9	40.41	76.779	*d* 62.928	1.220	Apr. 27
37	30	8.473	3.6	0	.....	.....	.....	Apr. 27

a Wind, 8.488 miles per hour.
b Wind, 8.500 miles per hour.
c Wind, 8.425 miles per hour.
d Wind, 8.478 miles per hour.

Wheels Nos. 35, 36, 37, and 38 contained 12 plain sails of lineal dimensions, as shown in fig. 19. Each sail made by fastening together 2 sails of No. 18. Wheels otherwise the same as No. 18, and differed from each other only in angles of weather.

Total area of sail surface, 12.937 square feet.

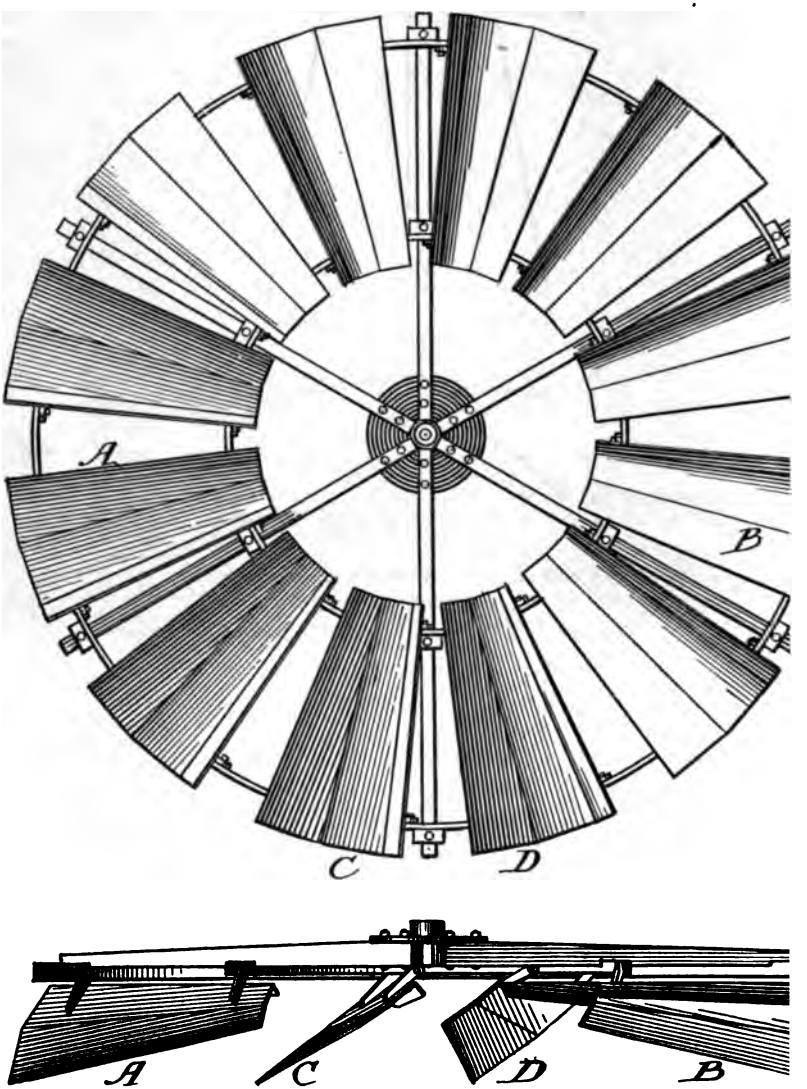


FIG. 22.—Elevation and section of wheel No. 39. (Sail area, 13.072 square feet; efficiency, 0

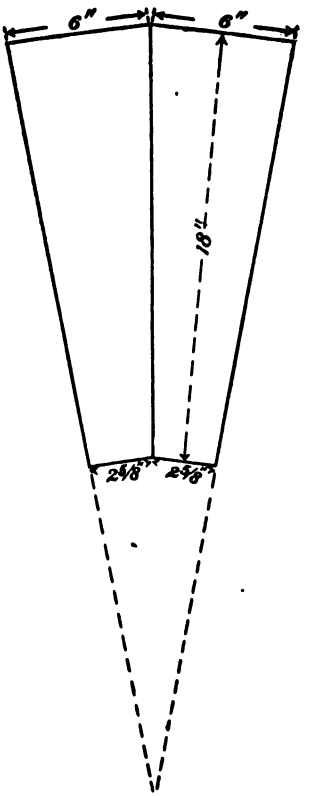


FIG. 19.—Dimensions of sails of wheels.
Nos. 35, 36, 37, and 38.

WHEELS NOS. 40 TO 43.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
41	25	8.461	0.1	84.70	.....	.....	.....	May 5
41	25	8.496	1.9	46.81	88.939	65.474=	1.358	May 5
41	25	8.496	3.45	0	.....	.....	.....	May 5
40	27.5	8.498	0.1	82.13	.....	.....	.....	May 3
40	27.5	8.459	1.9	45.74	86.906	62.928=	1.381	May 3
40	27.5	8.519	3.75	0	.....	.....	.....	May 3
42	30	8.491	0.1	77.95	.....	.....	.....	May 7
42	30	8.464	1.9	43.15	81.985	61.180=	1.339	May 7
42	30	8.475	3.9	0	.....	.....	.....	May 7
43	25, 30	8.528	0.1	84.92	.....	.....	.....	May 8
43	25, 30	8.476	1.9	45.92	87.248	62.396=	1.398	May 8
43	*25, 30*	*8.495*	*3.6*	0	.....	.....	.....	May 8

Wind for No. 2 the same as for wheel compared in each case. Wheels contained 12 angular concave sails of lineal dimensions as shown in fig. 21. Each sail made by fastening together two sails of No. 18 at an angle of 165 degrees so as to present concave surface to the wind. Wheels differed only in angles of weather. The sails of No. 43 were twisted, making the angles of weather 25 degrees at outer ends of sails and 30 degrees at inner ends. Total area of sail surface, 12.937 square feet.

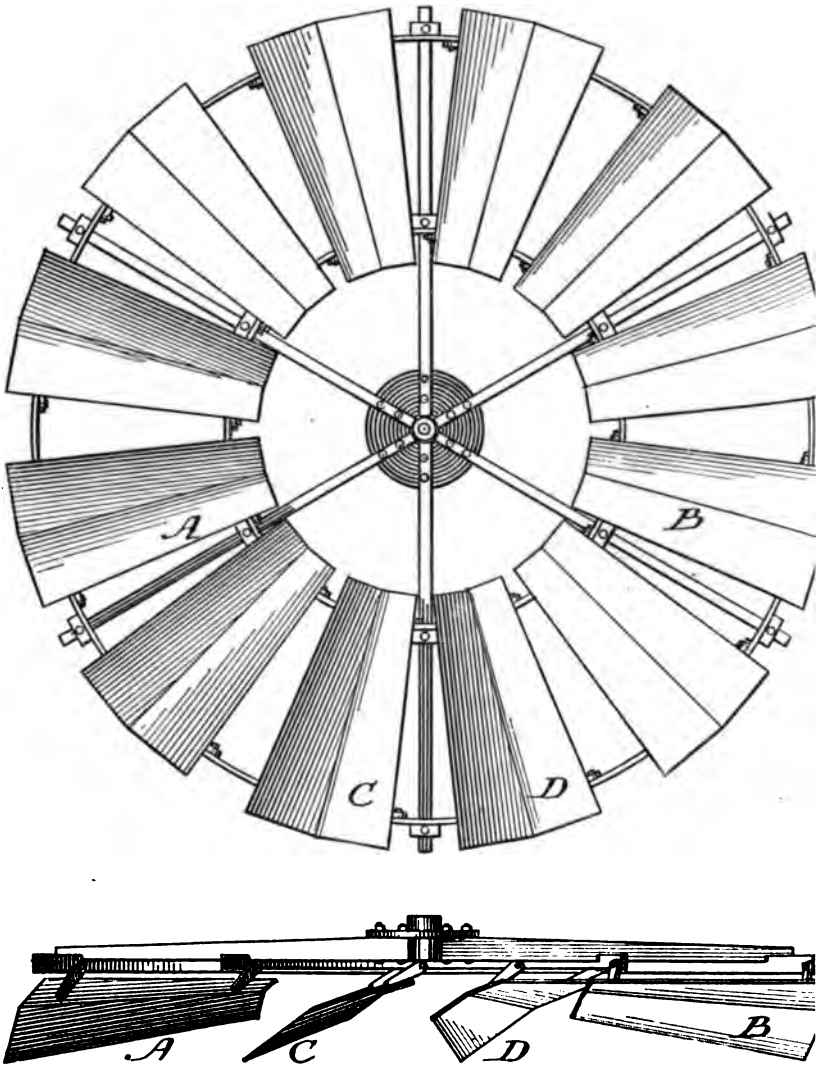


FIG. 20.—Elevation and section of wheel No. 40. (Sail area, 12.937 square feet; efficiency, 0.213.

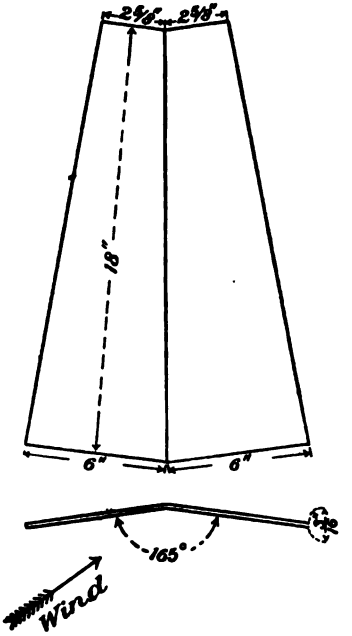


FIG. 21.—Dimensions of concave sails of wheels Nos. 40, 41, 42, and 43.

WHEELS NOS. 40 TO 42.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
41	25	6.444	0.1	65.33	.....	.....	.....	May 24
41	25	10.963	0.1	111.10	.....	.....	.....	May 29
41	25	6.459	1.1	35.07	38.577	28.270=	1.365	May 24
41	25	11.022	3.3	55.45	182.985	*a*137.346+	1.332	May 29
41	25	6.404	2.0	0	.....	.....	.....	May 24
41	25	11.111	5.6	0	.....	.....	.....	May 29
40	27.5	6.372	0.1	60.27	.....	.....	.....	May 14
40	27.5	10.890	0.1	106.73	.....	.....	.....	May 21
40	27.5	6.371	1.1	32.90	36.190	26.290=	1.377	May 14
40	27.5	11.041	3.3	56.32	185.856	135.498=	1.372	May 17
40	27.5	6.403	2.1	0	.....	.....	.....	May 14
40	27.5	10.944	6.3	0	.....	.....	.....	May 21
42	30	6.454	0.1	59.13	.....	.....	.....	May 23
42	30	10.974	0.1	103.50	.....	.....	.....	May 23
42	30	6.429	1.2	29.65	35.580	26.037=	1.367	May 23
42	30	10.997	3.7	50.62	187.294	138.897=	1.348	May 22
42	30	6.411	2.3	0	.....	.....	.....	May 23
42	*30*	*11.063*	*6.9*	0	.....	.....	.....	May 22

a Wind, 11.021 miles per hour.

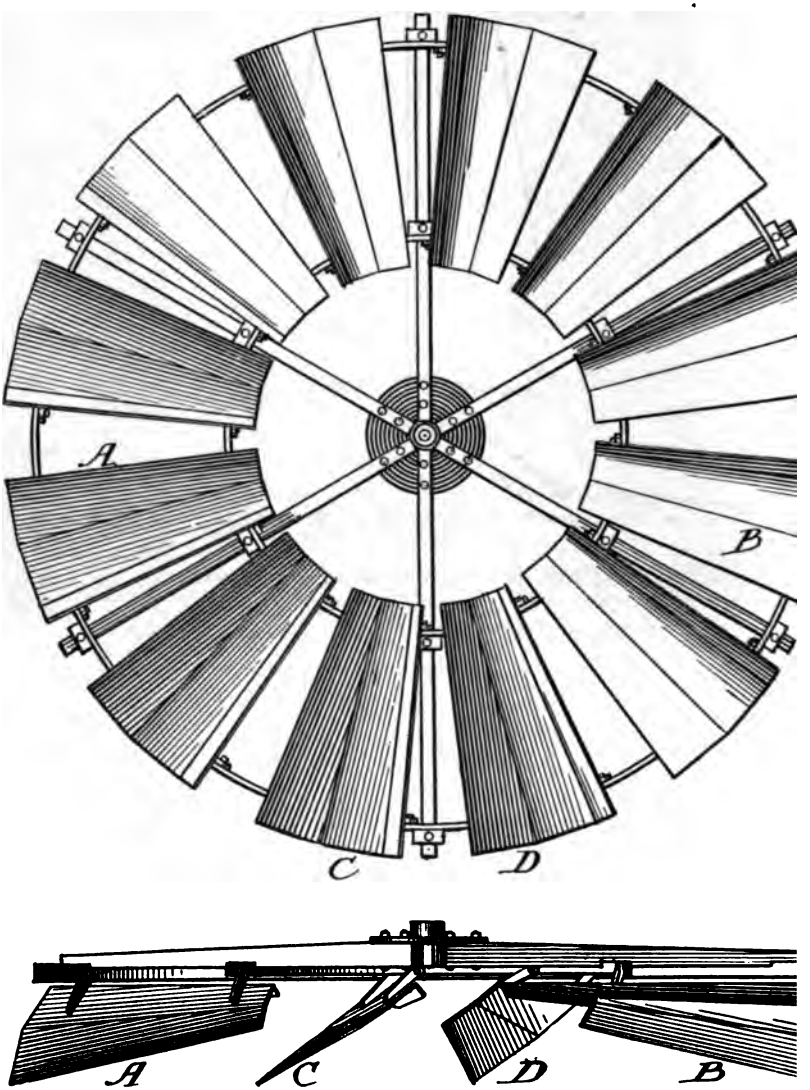


FIG. 22.—Elevation and section of wheel No. 39. (Sail area, 13.072 square feet; efficiency, 0.

WHEELS NOS. 35, 39, 40, AND 44.—*Results of experiments.*

. of eel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
35	27.5	8.413	0.1	76.78	.....	.....	.....	Apr. 24
35	27.5	8.424	1.7	46.64	79.288	*a*63.726—	1.244	Apr. 25
35	27.5	8.445	3.4	0	.....	.....	.....	Apr. 25
39	27.5	8.472	0.1	64.52	.....	.....	.....	May 1
39	27.5	8.512	1.9	35.57	67.583	63.650=	1.062	May 1
39	27.5	8.431	3.7	0	.....	.....	.....	May 1
40	27.5	8.498	0.1	82.13	.....	.....	.....	May 3
40	27.5	8.459	1.9	45.74	86.906	62.928=	1.381	May 3
40	27.5	8.519	3.75	0	.....	.....	.....	May 3
44	25, 30	8.482	0.1	84.88	.....	.....	.....	May 9
44	25, 30	8.477	1.9	44.35	84.265	60.952=	1.382	May 9
44	25, 30	8.475	3.6	0	.....	.....	.....	May 9

a Wind, 8.425 miles per hour.

Wheel No. 39 contained 12 sails of lineal dimensions as shown in fig. 23, made by ling to the sails of No. 35, along their rear edges, strips as illustrated, so as to sent concave surface to the wind. But the line *a b* was considered the face of the l as regards angle of weather. For No. 40 see previous description. Sails angu- concave.

Wheel No. 44 was the same as No. 3 except that the angles of weather were 25 grees at outer ends of sails and 30 degrees at inner ends. Sails circular concave.

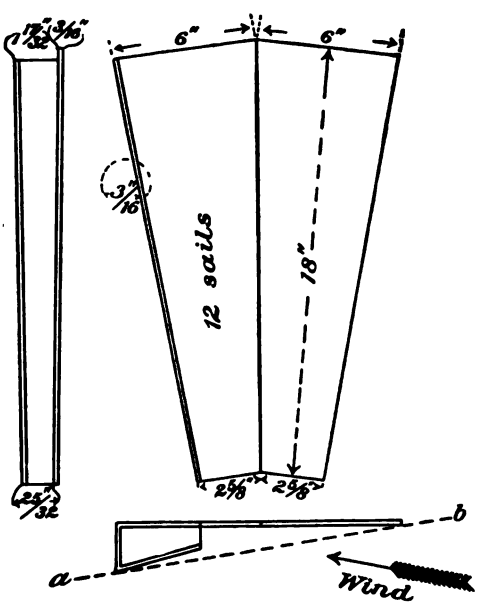


FIG. 23.—Dimensions of sails of wheel No. 39.

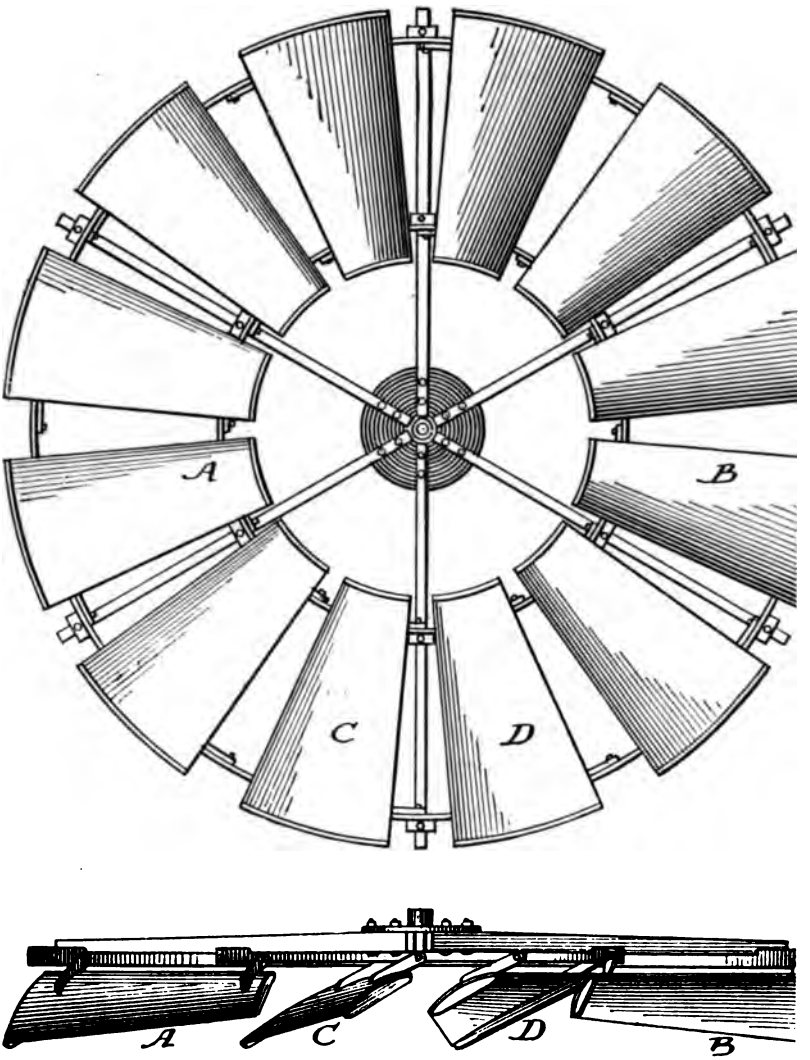


FIG. 24.—Elevation and section of wheel No. 44. (Sail area, 13.590 square feet; efficiency, 0.2

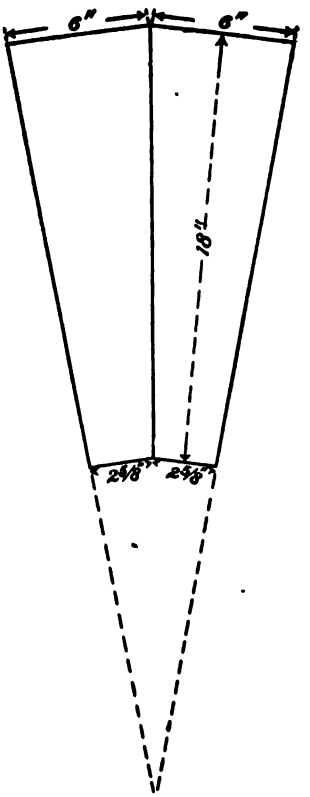


FIG. 19.—Dimensions of sails of wheels.
Nos. 35, 36, 37, and 38.

WHEELS NOS. 40 TO 43.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
41	25	8.461	0.1	84.70	.....	.....	.....	May 5
41	25	8.496	1.9	46.81	88.939	65.474=	1.358	May 5
41	25	8.496	3.45	0	.....	.....	.....	May 5
40	27.5	8.498	0.1	82.13	.....	.....	.....	May 3
40	27.5	8.459	1.9	45.74	86.906	62.928=	1.381	May 3
40	27.5	8.519	3.75	0	.....	.....	.....	May 3
42	30	8.491	0.1	77.95	.....	.....	.....	May 7
42	30	8.464	1.9	43.15	81.985	61.180=	1.339	May 7
42	30	8.475	3.9	0	.....	.....	.....	May 7
43	25, 30	8.528	0.1	84.92	.....	.....	.....	May 8
43	25, 30	8.476	1.9	45.92	87.248	62.396=	1.398	May 8
43	*25, 30*	*8.495*	*3.6*	0	.....	.....	.....	May 8

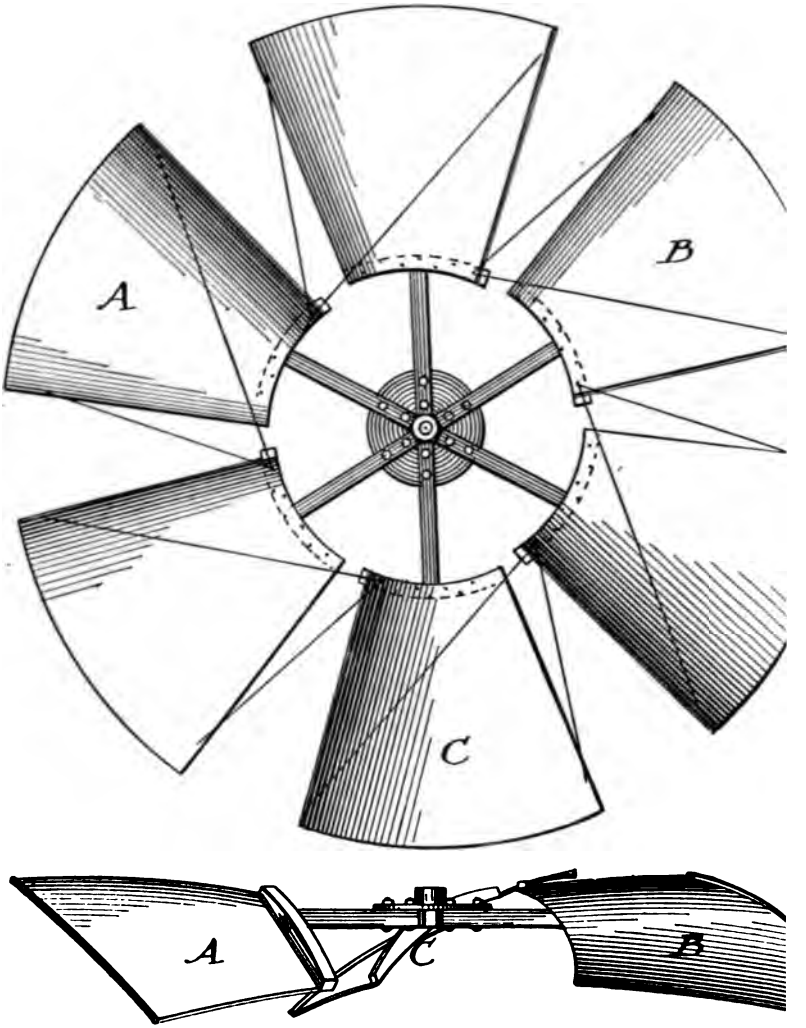


FIG. 25.—Elevation and section of wheel No. 48. (Sail area, 14.159 square feet; efficiency, 0.

Wheels Nos. 45 to 49 differed from each other only in angles of weather, and tained six sails of lineal dimensions as shown in fig. 26. Sails were made by g together several layers of paper which formed pasteboard about one-sixteen an inch thick. The backs of front and rear edges of each sail were bordered b pine strips. Backs of outer edges of sails were likewise bordered and held in by stiff curved pine pieces. The inner end of each sail was fastened securel thick curved pine piece, which was bolted against the outer end of a short ash so that the sail could be turned on the bolt and set at any desired angle of wea Thus the arms did not extend beyond the inner ends of the sails. The oute ners of each sail were held in position by two fine brass wires extending to the inner corner of the succeeding sail. The front and rear edges also curved *what, so that the sails* presented double concave surfaces to the wind. In cons *ing these wheels* we aimed at avoiding all unnecessary obstruction to the pass *wind between the sails.*

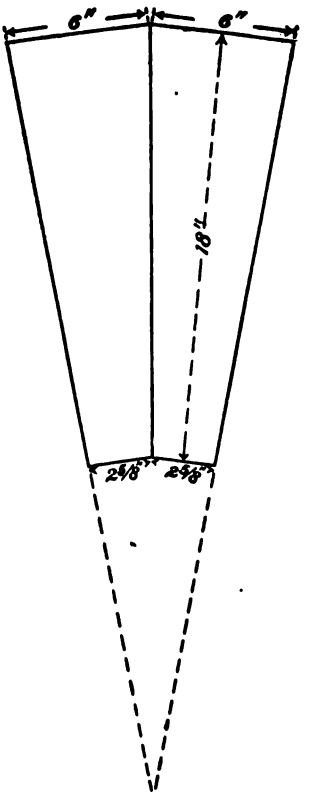


FIG. 19.—Dimensions of sails of wheels.
Nos. 35, 36, 37, and 38.

WHEELS NOS. 40 TO 43.—*Results of experiments.*

No. of wheel.	Angle of weather.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Date.
	°	*Miles.*	*Pounds.*					1883.
41	25	8.461	0.1	84.70	.....	.....	.....	May 5
41	25	8.496	1.9	46.81	88.939	65.474=	1.358	May 5
41	25	8.496	3.45	0	.....	.....	.....	May 5
40	27.5	8.498	0.1	82.13	.....	.....	.....	May 3
40	27.5	8.459	1.9	45.74	86.906	62.928=	1.381	May 3
40	27.5	8.519	3.75	0	.....	.....	.....	May 3
42	30	8.491	0.1	77.95	.....	.....	.....	May 7
42	30	8.464	1.9	43.15	81.985	61.180=	1.339	May 7
42	30	8.475	3.9	0	.....	.....	.....	May 7
43	25, 30	8.528	0.1	84.92	.....	.....	.....	May 8
43	25, 30	8.476	1.9	45.92	87.248	62.396=	1.398	May 8
43	*25, 30*	*8.495*	*3.6*	0	.....	.....	.....	May 8

rectangular piece of pine 1 by 1¼ by 17 inches was fastened to make an obstruction representing an extension of the arm, as illustrated in figs. 28 and 29.

The relative efficiency of No. 60 as compared with No. 61 is indicated by the equation

$$\frac{107.350}{72.447} = 1.482$$

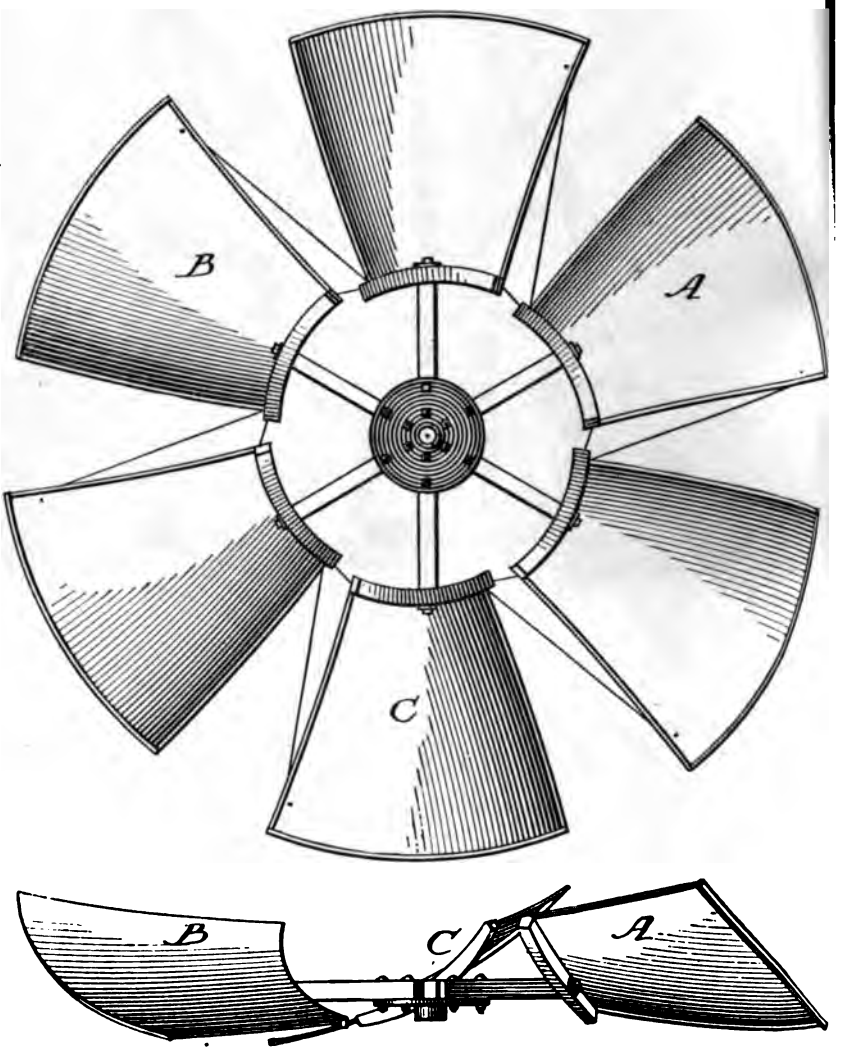


FIG. 27.—Elevation and section of wheel No. 60. (Sail area, 14.159 square feet; efficiency, 0.284.)

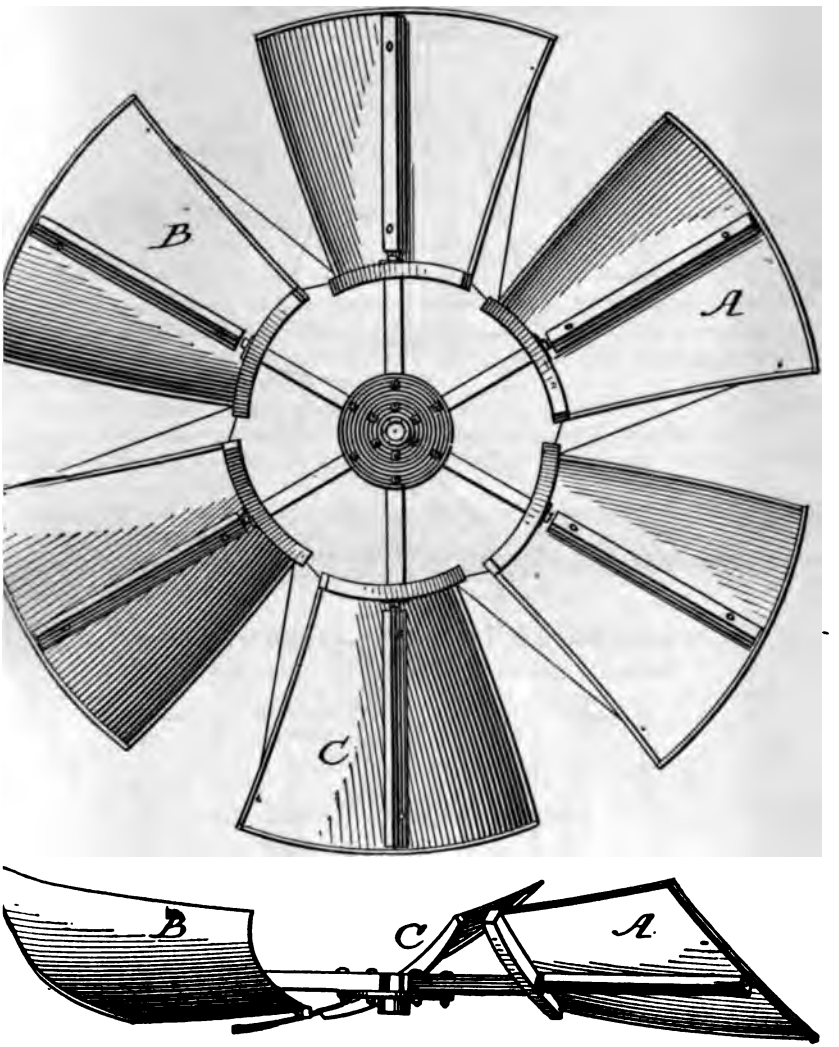


FIG. 28.—Elevation and section of wheel No. 61. (Sail area, 14.159 square feet; efficiency, 0.191.)

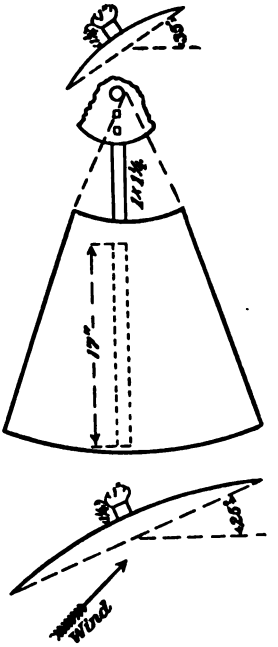


FIG. 29.—Dimensions of sails of wheel No. 61 with obstruction on back.

WHEELS NOS. 2, 49, AND 53 TO 57.—*Results of experiments.*

No of wheel.	Angle of deflector.	Velocity of wind per hour.	Loads applied +0.1 for friction.	Turns of wheel per minute.	Date.
	° ′	*Miles.*	*Pounds.*		1883.
54	30	10.989	0.1	79.40	July 3
53	45	11.156	0.1	81.80	July 3
55	60	10.985	0.1	80.52	July 3
2	.....	11.007	0.1	77.57	July 3
2	.....	10.975	3.3	37.83	July 5
56	50	10.968	3.3	40.63	July 5
57	50	11.054	3.5	62.63	July 5
49	.....	11.032	3.5	64.22	July 5

Wheels Nos. 53, 54, 55, and 56 were the No. 2 wheel, with stationary rectangular board, ¼ by 9⅜ by 22 inches, placed in front of sails and with its face set at various angles with plane of wheel, as indicated in table, to act as deflector. Board supported by arm outside of wheel on side farthest from axis of sweep; its lower edge at height of axis of wheel, horizontal, and parallel to plane of wheel. Inner end of board 9 inches from axis of wheel prolonged.

Wheel No. 57 was wheel No. 49, with the same board, described above, similarly used as deflector, except that its lower edge was placed so as to clear the forward edges of sails by about 2 inches at each end.

In the case of Nos. 53 to 56 the center of board deflector was about 7 inches from *center of sails.*

WHEELS NOS. 2, 49, 51, 52, 58, AND 59.—*Results of experiments.*

No. of wheel.	Position of obstruction.	Velocity of wind per hour.	Load applied +0.1 for friction.	Turns of wheel per minute.	Date.
		Miles.	*Pounds.*		1883.
2	.....	11.141	0.1	78.58	July 3
51	Before ..	11.130	0.1	75.73	July 3
52	Behind..	11.045	0.1	76.16	July 3
49	.....	10.966	3.5	64.53	July 5
58	Before ..	10.972	3.5	55.54	July 5
59	Behind..	10.972	3.5	60.06	July 5

el No. 51 was No. 2, with rectangular board ¼ by 9⅛ by 22 inches placed horiy in front of the side of wheel farthest from axis of sweep, its face parallel to f wheel so as to act as obstruction to wind. Board held at its outer end by of an arm outside of wheel.

ance from board to centers of sails, 8 inches.

ance from inner end of board to axis of wheel prolonged, 9 inches.

el No. 52 was the same as No. 51, except that the board was placed 8 inches centers of sails.

el No. 58 was No. 49, with obstrution board above described similarly placed, es in front of centers of sails.

el No. 59 was the same as No. 58, except that the board was placed 8¼ inches centers of sails.

SAILS AT BEST ANGLES OF WEATHER.

Efficiency of various wind wheels at best angles of weather.

Area of sail surface.	Angles of weather.	Wind per hour.	Turns of wheel unloaded.	Starting forces.	Load at maximum.	Turns of wheel at maximum.	Product at maximum.	Product of No. 2 at maximum.	Ratio of products.	Value per square foot of sail surface.	Number and indicated character of sails.
q. ft.	°	*Miles.*		*Lbs.*	*Lbs.*						
.190	35	8.452	60.0	.....	1.6	31.69	50.704	.....	.....	.....	30 a
.380	35	8.403	59.0	4.1	1.9	32.875	62.463	62.463	1.000	0.0544	60 a
.688	27.5	8.338	72.0	.....	1.7	39.33	66.861	.....	.....	.....	24 b
.937	27.5	8.500	75.5	3.2	1.9	39.57	75.183	62.719	1.204	.0931	24 c
.937	25, 35	8.502	76.5	3.4	1.7	43.77	74.409	61.655	1.207	.0938	24 c
.000	27.5	8.426	72.7	3.4	1.9	38.87	73.853	60.762	1.212	.0808	24 d
.016	27.5	8.406	73.0	3.6	1.9	39.10	74.290	61.845	1.201	.0706	24 e
.937	27.5	8.424	77.0	3.3	1.7	46.64	79.288	63.726	1.244	.0962	12 cc
.937	25	8.499	80.0	3.0	1.7	47.05	79.988	62.852	1.273	.0984	12 cc
.590	25, 30	8.477	85.0	3.5	1.9	44.35	84.265	60.953	1.382	.1017	12 f
.937	27.5	8.459	81.5	3.6	1.9	45.74	86.906	62.928	1.381	.1067	12 cc
.937	25, 30	8.476	84.5	3.5	1.9	45.92	87.248	62.396	1.398	.1081	12 cc
150	25, 27.5, 32.5, 35	10.935	141.0	6.1	3.7	66.79	247.123	132.033	1.872	.1322	6 g

The velocities of wind given in this table are strictly correct only for products at maximum, though they are very nearly correct also for products of No. 2 at maximum. The velocities of wind are only approximately correct for turns of wheels unloaded and for starting forces. Nothing was added to starting forces for friction, and the 0.1 pound was added to starting forces in preceding tables only for convenience.[1] For further particulars consult preceding tables.

RELATIONS OF THREE DIFFERENT VELOCITIES OF WIND.

Relation of different velocities of wind to maximum products.

Angle of weather.	No. of wheel.	Wind per hour.		Product at maximum.		$\frac{b}{a}$	$\left(\frac{b}{a}\right)^3$	$\frac{B}{A}$	$\frac{B}{A}-\left(\frac{b}{a}\right)^3$	$A\times\left(\frac{b}{a}\right)^3$
		a	b	A	B					
°		*Miles.*	*Miles.*							
	2	6.371	8.459	26.290	62.928	1.328—	2.342	2.394	+0.052	61.571
35	2	8.459	11.041	62.928	135.498	1.305+	2.223	2.153	— .070	139.889
	2	6.371	11.041	26.290	135.498	1.733	5.199	5.154	— .045	136.682
	40	6.371	8.459	36.190	86.906	1.328—	2.342	2.401	+ .059	84.757
27.5	40	8.459	11.041	86.906	185.856	1.305+	2.223	2.139	— .084	193.192
	40	6.371	11.041	36.190	185.856	1.733	5.199	5.135	— .064	188.152
	41	6.459	8.496	38.577	88.939	1.315+	2.274+	2.306	+ .032	87.724
25	41	8.496	11.022	88.939	182.985	1.297+	2.182	2.057+	— .125	194.065
	41	6.459	11.022	38.577	182.985	1.706+	4.965+	4.743	— .222	191.535
	42	6.429	8.464	35.580	81.985	1.317+	2.284+	2.304+	+ .020	81.265
30	42	8.464	10.997	81.985	187.294	1.299+	2.192	2.284+	+ .092	179.711
	42	6.429	10.997	35.580	187.294	1.707+	4.974	5.264	+ .290	176.975

Conclusion.—Maximum products vary as the cubes of the velocities of wind. Some allowance must be made for the differences in time and condition of air under which the various products were obtained; considering these differences, there is no unaccountable variation from the law of cubes.

Relation of different velocities of wind to starting forces.

Angle of weather.	No. of wheel.	Wind per hour.		Starting forces.		$\frac{b}{a}$	$\left(\frac{b}{a}\right)^2$	$\frac{B}{A}$
		a	b	A	B			
°		*Miles.*	*Miles.*	*Pounds.*	*Pounds.*			
	2	6.380	8.405	2.3	4.1	1.319—	1.740	1.783
35	2	8.405	10.989	4.1	7.0	1.307	1.708	1.704
	2	6.380	10.989	2.3	7.0	1.722	2.965	3.043
	40	6.403	8.519	2.0	3.65	1.330	1.769—	1.825
27.5	40	8.519	10.944	3.65	6.2	1.285	1.651	1.699
	40	6.403	10.944	2.0	6.2	1.709	2.921—	3.100
	41	6.404	8.496	1.9	3.35	1.327—	1.761	1.763
25	41	8.496	11.111	3.35	5.5	1.308—	1.711—	1.642
	41	6.404	11.111	1.9	5.5	1.735	3.010	2.895
	42	6.411	8.475	2.2	3.8	1.322	1.748	1.727
30	42	8.475	11.063	3.8	6.8	1.305—	1.703	1.789
	42	6.411	11.063	2.2	6.8	1.725	2.976—	3.091

Conclusion.—Starting forces vary as the squares of the velocities of wind.

[1] See starting forces, "S," p. 25.

Relation of different velocities of wind to speed of unloaded wheels.

Angle of weather.	Nos. of wheels.	Wind per hour.		Turns unloaded.		$\frac{b}{a}$	$\frac{B}{A}$
		a	*b*	*A*	*B*		
°		*Miles.*	*Miles.*				
35	2	6.437	8.511	43.78	59.62	1.322	1.362
	2	8.511	10.996	59.62	78.68	1.292	1.320
	2	6.437	10.996	43.78	78.68	1.708	1.797
27½	40	6.372	8.498	60.27	82.13	1.334	1.363
	40	8.498	10.890	82.13	106.73	1.281	1.300
	40	6.372	10.890	60.27	106.73	1.709	1.771
25	41	6.444	8.461	[1] 65.33	84.70	1.313	1.296
	41	8.461	10.963	84.70	111.10	1.296	1.312
	41	6.444	10.963	[1] 65.33	111.10	1.701	1.701
30	42	6.454	8.491	59.13	77.95	1.316	1.318
	42	8.491	10.974	77.95	103.50	1.292	1.328
	42	6.454	10.974	59.13	103.50	1.700	1.750

[1] Too great because of outside wind and perhaps other causes.

Conclusion.—Speeds of unloaded wheels increase in somewhat greater ratio than the velocities of wind.

All wind wheels tested were 5 feet in external diameter, and all sails were 18 inches long except in the case of Nos. 46 to 49 and Nos. 60 and 61, in which wheels the sails were 19 inches long, 1 inch being added for support at inner ends.

AUTOMATIC BRAKE ADJUSTER.

On March 13, 1883, the dynamometer was improved by the addition of an automatic brake adjuster. Previous to this date the brake was adjusted as described on page 23 through regulating the tension of the adjusting cord by sliding a weight in and out on a lever attached to the cord.

This method required time and patience in order to get an exact balance between the load applied and the friction of brake, and it was also necessary to exercise judgment in determining when the balance was exactly even. Furthermore, an exact balance was not always retained during a test of one minute, as required for accuracy, and frequent readjustments were constantly necessary. The labor involved in adjusting and maintaining a proper balance of friction and load became very tiresome when kept up for from six to eight hours continuously, as was frequently the case; and weariness was detrimental to the perceptions as well as to the judgment. Therefore, an automatic adjuster became very desirable, both for relief to the mind and for the increase of accuracy.

The automatic brake adjuster, after various attempts and failures, was finally constructed as follows: The brake and adjusting cord were

left substantially as previously described on page 23; but one end of a short rod three-sixteenths inch in diameter was attached to the cord where it was brought back from the brake in a direction parallel to the axis of the shaft. The other end of this rod had a full thread, 32 to the inch, and a nut which was fastened to a wooden wheel *A*, 6½ inches in diameter. This wheel was supported on a hollow tube through which the rod passed, and against the outer end of which the nut rested. The nut communicated tension to the adjusting cord, which was tightened by revolving the wheel *A* in one direction and loosened by turning the wheel in the opposite direction. This wheel *A* was driven by a small wheel *B*, 1½ inches in diameter, fastened to a large wheel *C*, 5 inches in diameter. This wheel *C* was driven by another small wheel *D*, 1¼ inches in diameter, fastened to a large wheel *E*, 4 inches in diameter, the periphery of which was constantly in contact with another wheel *F* of the same dimensions. The wheels *E* and *F* were supported by a tilting frame which oscillated on a supporting rod passing through the centers of the wheels *B* and *C*, but not touching them. The wheels *E* and *F* hung in close proximity to the rear end of wind wheel shaft, 1 1/16 inches in diameter, so that a slight movement of the tilting frame would bring either of the wheels *E* or *F* in contact with the shaft; and if wheel *E* came in contact with the shaft while revolving, the adjusting cord would be drawn tighter, and if wheel *F* came in contact with the shaft, the tension of the cord would be relaxed. The tilting frame was connected with the brake and partook of its oscillating motion, so that if the friction of the brake was too small to lift the load applied, wheel *E* would be in contact with the shaft and would cause an increase in the friction of the brake until the load was lifted and contact between wheel *E* and the shaft broken. If the friction of the brake was too great, the load applied was raised until wheel *F* came in contact with the shaft so as to cause a diminution of friction until contact of wheel *F* with the shaft was broken. It was very easy to tell at any time if friction of brake was too little or too great, by simply watching the direction of movement of wheels *E* and *F*. This did not require the exercise of judgment, as errors of balance were greatly magnified to the eye and made self-correcting. All contacts between the various wheels, *A*, *B*, *C*, *D*, *E*, *F*, and shaft were frictional. Wheels *B*, *C*, were entirely supported by contact of peripheries with wheels *A* and *D*, and wheel *E* was so hung to a separately hinged support that its own weight produced a uniform pressure against wheel *F*. The entire automatic system was supported independently of the brake; the connection with the brake served only to give the necessary oscillatory movement to the tilting frame, which was also balanced both as regards the action of gravity and centrifugal force when the sweep was in motion.

Some power of the wheel was necessarily consumed by the automatic adjuster, but it may be ascertained, from the description and *dimensions given*, that a point in the wind wheel 1 foot from the center

of shaft would move in tightening the adjusting cord about 67,019 times as far as the threaded rod attached to the cord; from this it follows that the pressure of the brake against the cylinder was $67{,}019 \times 16 = 1{,}072{,}304$ times as great as the force required at 1 foot from the center of shaft to produce the pressure, without considering the friction of the automatic system. And if we make the very liberal allowance of one-third additional force required to overcome the friction of the automatic system and call the friction of brake 0.05 of pressure, remembering that the diameter of cylinder was $5\frac{1}{4}$ inches, we may readily find that the force required to produce friction of brake equal to any load would be only about $\frac{1}{13064}$ part of that load at the same distance from center of shaft. Hence, it will be seen that the power consumed by the automatic brake adjuster was too minute materially to affect products and called for no correction.

THEORY OF THE ACTION OF THE WIND UPON THE SAILS OF WINDMILLS.

Where a fluid moves along in a current, the direct force of the current is expressed in pounds by the formula Mv, in which v represents the velocity of flow in feet per second and M is the mass of fluid which flows per second past a fixed point. $M = \frac{W}{g}$, W representing weight in pounds of fluid passing the fixed point per second, and $g = 32.2$, the constant for gravity. [1]

Where a current acts against a surface and its direction is simply changed by the surface without impeding its velocity, a direct pressure $= Mv$ is exerted by the impinging current in the direction of its motion, and an equal pressure is exerted by the current as it escapes from the surface, $= Mv$, in a direction opposite to the direction of escape.

If the current impinges against a surface which is itself in motion, the relative velocities of the impinging and escaping currents must be considered in determining the forces which act upon the moving surface. Of the two equal forces due to impingement and escape of current the sum of those components acting in the direction of motion of the surface constitutes the useful effort designated by P, and if the velocity per second of motion of surface is called u, the useful work yielded per second by the current will be represented by Pu. The actual energy of the current is represented [2] by $\frac{Mv^2}{2} = \frac{Wv^2}{2g}$. The ratio of useful work to actual energy is termed efficiency, and is represented [3] by the symbol $1 - k = \frac{2Pug}{Wv^2}$.

In deducing formulæ to represent the action of wind upon the sails of windmills we have proceeded upon the assumption that the sail

[1] See Rankine's Steam Engine, articles 14 and 144, Case III.
[2] See Rankine's Steam Engine, article 31.
[3] See Rankine's Steam Engine, article 92.

should be of such form as most effectually to change the air current into a direction as nearly as possible opposite to the direction of the sail's motion. We have also assumed, in accordance with principles stated by Rankine concerning the action of water on vanes, that the receiving edge of the sail should be tangent to the relative direction of the impinging current. These conditions are represented in fig. 30, in which $DE = v =$ direction and velocity of wind, $AE = u =$ direction and velocity of sail, and $\alpha =$ angle DEA, which the direction of wind makes with direction of motion of sail.

Then, $DA =$ relative direction and velocity of current with reference to sail AB.

Let $BG = AD =$ relative direction and velocity of current as it escapes from the sail.

$\beta =$ angle BGF which relative direction of escaping current makes with direction of sail's motion.

Then

$$DC = v \sin \alpha.$$
$$AC = u - v \cos \alpha.$$
$$AD = \sqrt{v^2 \sin^2 \alpha + (u - v \cos \alpha)^2}.$$
$$GF = GB \cos \beta = AD \cos \beta.$$
$$GF = \cos \beta \sqrt{v^2 \sin^2 \alpha + u^2 - 2uv \cos \alpha + v^2 \cos^2 \alpha}.$$
$$GF = \cos \beta \sqrt{v^2 + u^2 - 2uv \cos \alpha}.$$

If $Q =$ volume in cubic foot of air acting on sail in 1 second, and $d =$ weight in pounds of 1 cubic foot of air, we have for useful effort

$$P = \frac{dQ}{g}(GF - AC).$$
$$P = \frac{dQ}{g}[\cos \beta \sqrt{u^2 + v^2 - 2uv \cos \alpha} - u + v \cos \alpha].$$

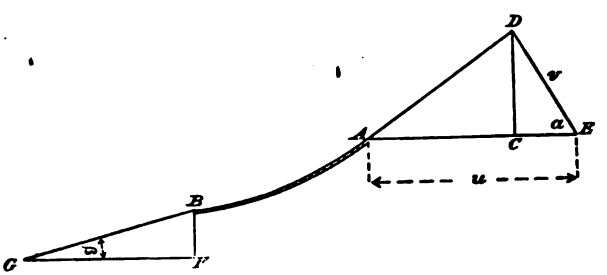


FIG. 30.—Action of wind on a sail.

The expression for useful work becomes

$$Pu = \frac{dQ}{g}[u \cos \beta \sqrt{u^2 + v^2 - 2uv \cos \alpha} - u^2 + uv \cos \alpha]. \qquad \text{(A)}$$

Dividing by $\frac{Mv^2}{2} = \frac{dQv^2}{2g}$ we have for efficiency

$$1 - k = 2\frac{u}{v} \cos \beta \sqrt{1 + \frac{u^2}{v^2} - 2\frac{u}{v} \cos \alpha} - 2\frac{u^2}{v^2} + 2\frac{u}{v} \cos \alpha.$$

$\frac{u}{v}=x,$

$$1-k=2x\cos\beta\sqrt{1+x^2-2x\cos\alpha}-2x^2+2x\cos\alpha. \qquad \text{(B)}$$

tituting $-\cos\gamma$ for $\cos\beta$ in formula (A) the expression for
s the same as formula (6) on page 165 of Rankine's Steam

an expression for x corresponding to maximum efficiency, for
lues of α and β would, as Rankine remarks, be more trouble
worth. But in the case of the windmill we know that
ich makes $\cos\alpha=0$. Hence, formula (B) becomes

$$1-k=2\cos\beta\sqrt{x^2+x^4}-2x^2. \qquad \text{(C)}$$

tion between x and β corresponding to maximum and mini-
ency will be expressed by the formula

$$x^2=-\tfrac{1}{2}\pm\tfrac{1}{2}\sqrt{\frac{1}{1-\cos^2\beta}}=-\tfrac{1}{2}\pm\frac{1}{2\sin\beta}. \qquad \text{(D)}$$

s it appears that if $\beta=0$, as it should for maximum efficiency,
infinite. That is, the theoretical efficiency of the windmill
eater as the speed of the sails increases indefinitely.
ormula (C) shows that

For $x=1$, $1-k=.828$; for $x=2$, $1-k=.944$;
For $x=3$, $1-k=.973$; for $x=10$, $1-k=.9974$;

erfect efficiency when the speed of the sails is 10 times the
the wind. Perfect efficiency, however, could not be reached
o.
e the formula does not take into account the retarding effect
, resistance of medium, and other causes which make it
to realize in practice anything like the efficiency indicated
ula; and in practice β can never equal 0 as it should for
ciency.
$x=1$ and assigning more probable values to β, formula (C)

For $\beta=10°$, $1-k=.785$.
For $\beta=20°$, $1-k=.657$.
For $\beta=30°$, $1-k=.449$.

ally the analysis of the action of wind on the sails of a wind-
ponds to the analysis of the action of water on a reaction
rbine without guide blades.[1] In the windmill, as commonly
e α in our formula (B) is necessarily 90°; but the formula
ide blades or deflectors corresponding to those of parallel-

[1] See Rankine's Steam Engine, pp. 197 and 206.

flow turbine water wheels. For if we give to x and β any probable finite values, we may find the values of angle α corresponding to maximum efficiency from the formula,

$$\cos \alpha = \tfrac{1}{2}\left(x + \frac{1}{x} - x \cos^2 \beta\right) \qquad \text{(E)}$$

in which—

For $x = 3$ and $\beta = 10°$, $\alpha = 77° \; 46'$;
For $x = 2$ and $\beta = 10°$, $\alpha = 73° \; 44'$;
For $x = 3$ and $\beta = 20°$, $\alpha = 70° \; 00'$;
For $x = 2$ and $\beta = 20°$, $\alpha = 68° \; 28'$;
For $x = 1$ and $\beta = 20°$, $\alpha = 56° \; 03'$.

These assumptions for x and β are plausible, as x varies greatly between the outer and inner extremities of a sail, and β should be assumed as the average angle of escape of the whole cylinder of air intercepted by the wheel, and not as the angle merely of that portion of air which comes immediately in contact with the sails, or which the sails directly intercept.

It is to be noticed that formula (B) does not take into account as such the angle of weather, which seems to be incidental to practice but not to theory.

As we have already pointed out, our formula (A) is substantially the same as Rankine's general formula for the action of water on vanes. But Rankine, instead of applying his general formula to windmills, breaks entirely away from it, and uses another formula specially deduced for the "case of a flat vane oblique to the jet," and which he calls the "easiest method of solution."[1] But that his "easiest method" is erroneous even in the case specified, is evidenced by the fact that a very different result is obtained by the application of his general formula.

We pointed out the nature of Rankine's error in a paper dated December, 1881, which was submitted to Prof. R. H. Thurston, of Stevens Institute of Technology. Other writers on windmills have made the same mistake. The theory and formula which we have here given are abstracted from a paper also prepared for Professor Thurston, dated March 6, 1882.

Theoretical considerations were kept in view during our dynamometric experiments. It will be seen from the preceding tables of experiments how closely the results obtained in practice correspond to theory.

DISCUSSION OF RESULTS.

THERMOMETRIC AND BAROMETRIC INFLUENCES.

The preceding pages are substantially an unchanged reproduction of the records made by the writer for the United States Wind Engine and Pump Company of Batavia, Illinois, during and immediately after the experiments in 1883. The only alteration we have presumed to

[1] See Rankine's Steam Engine, pp. 169 and 215.

nake is in omiting the records of barometric readings which were made luring the earlier experiments. No barometric readings were taken or recorded after December 21, 1882.

CONSTRUCTION OF TABLES.

In tables on pages 28 to 46 results are recorded for a greater or less number of loads increasing usually by increments of two-tenths pound. Although the results are always given in the order of increasing loads, they were not always obtained in that order, as is indicated by the marginal dates at the right-hand side of some of the tables. Enough tables were prepared in this manner to show how in general the speeds of revolution decreased as the loads increased, and how the products increased with the increasing loads up to a certain point, which marks the maximum, and then decreased continually with further increments of load until the vanishing point was reached. It was not necessary to try many different loads in order to find the best load corresponding to the maximum product. This could be ascertained by a few trials, and as our main object was to get at the relative merits of the wheels tested, we subsequently abandoned the practice of trying so many different loads, and confined our attention more strictly to the determination of maximum products, starting forces, and unloaded speeds; so that the remaining tables on pages 47 to 65 show in general only these three results for each wheel and comparisons of results with different wheels. Special explanations relative to special tables are given elsewhere.

EXTREME RESULTS.

In addition to the original tables already presented, we have collected together in the following table the maximum products given by wind wheel No. 2 at the various dates it was tested in connection with other wheels. It may be observed that the lowest maximum product was 58.083, recorded on April 10, 1883, and that the highest product for wind of nearly the same velocity was 65.474, recorded on May 5, 1883. The products for the same wheel thus vary nearly 13 per cent on different days. Taking the temperature and barometric pressure of Chicago on the two days above mentioned as approximately the same as at Batavia, 35 miles distant due west, we are able to account for but little more than 2 per cent of the variation. Notice also that in wind of about 11 miles per hour, the lowest maximum product was 124.839, recorded on July 5, 1883, and that the highest product was 138.897, recorded on May 22, 1883. Here a variation of over 11 per cent appears for the same wheel on different days. In this case the difference in temperature and barometric pressure at Chicago might account for a little more than 8 per cent of the variation.

Maximum product of wheel No. 2 at different dates.

Velocity of wind per hour.	Product at maximum.	Date.	Velocity of wind per hour.	Product at maximum.	Date.
Miles.		1883.	*Miles.*		1883.
8.485	58.083—	Apr. 10	8.478	62.928—	Apr. 27
8.499	62.719+	Apr. 11	8.496	65.474=	May 5
8.487	62.966—	Apr. 12	8.459	62.928=	May 3
8.463	62.225	Apr. 16	8.464	61.180=	May 7
8.461	58.957—	Apr. 17	8.476	62.396=	May 8
8.432	62.966+	Apr. 9	8.512	63.650=	May 1
8.432	60.762	Apr. 6	8.477	60.952	May 9
8.455	61.902—	Apr. 6	8.452	58.273	Sept. 15
8.398	61.142	Apr. 4	11.041	135.498	May 17
8.395	61.370+	Apr. 3	11.054	126.984	July 2
8.403	62.463—	Mar. 14	11.021	137.346	May 29
8.410	61.845—	Mar. 27	10.997	138.897	May 22
8.427	62.478+	Mar. 28	10.980	135.201	June 25
8.419	61.864—	Mar. 30	11.000	129.195	June 20
8.446	61.408+	Mar. 31	10.980	132.264	June 26
8.501	61.655+	Apr. 18	10.935	132.033	June 28
8.488	63.802	Apr. 28	10.994	127.116	June 29
8.500	62.852—	Apr. 26	10.975	124.839	July 5
8.425	63.726—	Apr. 25			

INFLUENCE OF OUTSIDE WIND.

Experiments were conducted in a closed room to insure the exclusion of all natural wind. The exclusion of wind from the inclosed space, however, was not perfect. The wind circling around the outside of the building undoubtedly at times caused a slight circulation of air inside, which may have been more potent in causing variations in results than either thermometric or barometric influences. The windows were always kept carefully closed during experiments, and the doors also were guarded as much as possible. The doors, however, were necessarily used, and we distinctly remember being obliged to discard many measurements on account of doors being opened during tests on windy days. But unless a door was opened during a test we considered that errors from influence of outside wind were sufficiently guarded against by our practice of comparing all results with those obtained with wheel No. 2, under like conditions, as explained in the original records.

STANDARD OF COMPARISON.

As stated in the original record, wheel No. 2, which we always used as a standard for comparison, was modeled after the Halliday 10-foot wind wheel; that is to say, the slats, arms, etc., were the same in number

and of the same relative dimensions on a scale of one-half. There was, however, this difference, that the Halliday was what is called a section wheel, having pivoted sections for the purpose of regulation. Wheel No. 2 had all its parts rigidly fixed with reference to each other, and therefore was more like the solid wheels in common use at the time. The solid wheels were really copies of the Halliday, so far as the sails or slats and the arms were concerned. The only essential difference between section and solid wind wheels related to the method of stopping and governing their motion. Different makers of both styles of wheels had their own notions about angles of weather and proportions of sail surface. The Halliday 10-foot wheel had its slats set at an angle of 35° with the plane of motion. The slats were as thin as was consistent with safety, considering that their material was wood, and they were trimmed to a sharp edge, so as to cut the air as much as possible.

Nearly all wind wheels in use at the time of our experiments in 1882–83 were made with narrow wooden slats, similar in general appearance to our wheel No. 2. Some makers adopted greater angles of weather than 35 degrees, and even let the sails overlap so as to boast of greater sail area for their wheels as a basis for claiming greater power. A 45-degree angle of weather was not uncommon, and the slats were often made thicker than those of the Halliday wheel; besides, they were not always trimmed to an edge in front.

In adopting as our standard of comparison a wheel modeled after the 10-foot Halliday, we believe that we made use of the best model available to represent wind wheels in general as then constructed.

CORRECTIONS FOR AXLE FRICTION.

In the earlier tables 0.3 pound was added to all applied loads. This 0.3 pound was arrived at as explained after the table on page 38, and was in reality an estimate of frictional resistance to starting, or to slow motion. As authorities stated that friction was the same at all velocities, we adopted 0.3 pound as approximately representing the frictional resistance of the journals without regard to speed. We afterwards learned that the axial friction diminished greatly with increase of speed of revolution, as is made evident by comparing the speeds of the same wheels after starting friction was greatly reduced with the speeds before reduction of friction under the same applied loads and also unloaded. (See original records after table on page 38.)

Wheels Nos. 2, 18, and 19, especially, may be compared. Subtract 0.3 from loads in the tables giving average results for wheels Nos. 2, 18, and 19, on pages 29, 43, and 44, and subtract 0.1 in tables on pages 27 and 47 to obtain applied loads. Evidently there is very little difference between the axial frictions in the two cases for unloaded wheels.

We can not now, after a lapse of more than fifteen years, undertake to assert *what the axial friction* previous to March 13, 1883, should *have been called for different* speeds. But if in the table on page 29 we

call friction 0.15 pound, we shall find the maximum product to be 61.846 instead of 66.860, as there recorded. And as this correction would make this table agree substantially with the general results of the table on page 74, it seems probable that 0.15 pound is about the right amount to allow for friction in the first twenty-one tables to obtain maximum products.

If in the table on page 43 for wheel No. 18 we call friction 0.15 pound, the maximum product will become 76.869, obtained January 13, 1883. On April 10, 1883, the same wheel, No. 18, gave as maximum product 69.300, showing a variation of more than 12 per cent. But the United States Signal Service records in Chicago show for these two dates a difference of more than 1 inch in the barometric column, and about 39 degrees difference in temperature, which will account for a variation of about 12 per cent. Again, if in the table on page 44 for wheel No. 19 we call friction 0.15 pound, the maximum product will become 82.130, obtained January 18, 1883. On April 11, 1883, the same wheel, No. 19, gave as maximum product 75.185, showing a variation of over 9 per cent. The differences in temperature and barometric pressure as given for these two dates by the Signal Service records at Chicago would account for a variation in atmospheric density of just about 9 per cent. These three wheels, Nos. 2, 18, and 19, are the only ones tested both before and after the reduction of friction on March 13, 1883. While the products as recorded in the tables in which 0.3 pound was added to loads are all too large, their comparative values are not greatly distorted by the excessive allowance for axial friction.

The smaller allowance for axial friction will sometimes make the maximum product correspond to the next greater applied load, as in the case of wheel No. 19, page 44. But in the tables on pages 29 and 43, after allowing 0.15 pound instead of 0.3 pound for friction, the maximum products still correspond to the same applied loads and the same speeds of revolution as before.

AERIAL FRICTION.

So far we have not attempted to estimate the friction and resistances of the air itself, although the tables on pages 57, 59, and 61 are very suggestive on this point, and theoretical considerations also indicate the great importance of this subject, as will hereafter appear.

RELATION BETWEEN SPEED AND LOAD.

The following table shows the relation between speed and load for various wind wheels:

Table showing the relation between speed and load.

No. of wheel.	Angles of weather.	A	B	$\frac{A}{B}$	C	D	$\frac{C}{D}$	Sail surface.
		Turns per minute at maximum.	Turns per minute unloaded.		Load at maximum.	Starting force.		
	°							*Square feet.*
1	35	31.69	59.08	.54	1.65	2.55	.65	9.19
2	35	33.43	57.95	.58	1.85	3.55	.52	18.38
3	24.5, 11.25	47.48	90.48	.52	1.35	2.10	.64	13.59
4	20, 30	55.93	89.43	.63	1.55	3.00	.52	13.59
5	22.5, 32.5	46.10	86.78	.53	1.95	3.4	.57	13.59
6	30	42.10	69.03	.61	1.65	3.1	.53	10.688
7	25	47.40	73.60	.64	1.35	2.5	.54	10.688
8	35	36.84	62.67	.59	1.75	3.25	.54	10.688
9	32.5	34.58	66.40	.52	1.95	3.1	.63	10.688
10	27.5	39.33	72.23	.54	1.75	2.95	.59	10.688
11	20	45.46	77.60	.59	1.35	2.00	.68	10.688
12	15	43.88	75.60	.58	1.15	1.4	.82	10.688
13	40	28.70	56.33	.51	2.15	3.4	.63	10.688
14	45	24.72	48.60	.51	2.15	3.5	.61	10.688
15	47.5	25.30	46.84	.54	1.95	3.65	.53	10.688
16	50	20.67	42.40	.49	2.15	3.60	.60	10.688
17	25, 35	38.70	73.90	.52	1.75	2.75	.64	10.688
18	25	39.42	75.58	.52	1.95	3.1	.63	12.937
19	27.5	38.20	72.26	.529	2.15	3.3	.65	12.937
20	30	39.87	70.78	.56	1.9	3.45	.55	12.937
21	32.5	38.72	67.97	.57	1.9	3.6	.53	12.937
22	35	31.96	64.20	.50	2.1	3.8	.55	12.937
23	25, 30	43.21	73.46	.59	1.75	3.25	.54	12.937
24	27.5	38.87	72.60	.54	1.8	3.4	.53	15.000
25	30	38.95	69.00	.56	1.8	3.6	.50	15.000
26	25	43.74	75.38	.58	1.7	3.15	.54	15.000
27	35	33.74	63.65	.53	2.1	4.0	.53	15.000
28	32.5	34.49	67.07	.51	2.1	3.8	.55	15.000
29	27.5	39.10	73.10	.53	1.9	3.6	.53	17.016
30	30	35.15	69.18	.57	2.1	3.9	.54	17.016
31	32.5	34.94	66.73	.52	2.1	4.0	.52	17.016
32	35	33.43	62.90	.53	2.1	4.2	.50	17.016
33	25	37.80	75.46	.50	1.9	3.4	.56	17.016
34	25, 35	43.77	76.47	.57	1.7	3.4	.50	12.937
35	27.5	46.64	76.78	.61	1.7	3.3	.52	12.937
36	25	47.05	80.67	.58	1.7	3.0	.57	12.937
37	30	40.41	73.12	.55	1.9	3.5	.54	12.937
38	22.5	52.51	84.65	.62	1.5	2.8	.54	12.937
39	*27.5*	*35.57*	64.52	.55	1.9	3.6	.53	13.072

Table showing the relation between speed and load—Continued.

No. of wheel.	Angles of weather.	A	B	$\frac{A}{B}$	C	D	$\frac{C}{D}$	Sail surface.
		Turns per minute at maximum.	Turns per minute unloaded.		Load at maximum.	Starting force.		
	°							*Square feet.*
40	27.5	45.74	82.13	.56	1.9	3.65	.52	12.937
41	25	46.81	84.70	.55	1.9	3.35	.53	12.937
42	30	43.15	77.95	.55	1.9	3.8	.50	12.937
43	25, 30	45.92	84.92	.54	1.9	3.5	.54	12.937
44	25, 30	44.35	84.88	.52	1.9	3.5	.54	13.590
45	20, 30	79.99	156.63	.51	2.9	5.1	.57	14.159
46	17.5, 27.5	85.19	160.90	.52	2.7	4.7	.57	14.159
47	22.5, 32.5	73.31	148.68	.49	3.3	5.9	.56	14.159
48	25, 35	66.79	141.97	.47	3.7	6.4	.58	14.159
49	27.5, 37.5	58.82	136.30	.43	3.9	6.5	.60	14.159

BEST SPEEDS FOR WIND WHEELS.

In the fifth column of the preceding table the decimal fractions $\frac{A}{B}$ indicate for the various wind wheels the ratio of speeds at maximum work to the speed of wheels unloaded.

By consulting the original tables on pages 28 to 46, it may be noticed that there are generally four or five products nearly as great as the maximum product for each wheel. Where the two greatest products are nearly the same, a very slight difference in velocity of wind, or other cause, may make the maximum result correspond to either of two different loads or two different speeds. This may account for certain apparently rather wide variations in the columns $\frac{A}{B}$ and $\frac{C}{D}$.

In general the best speed for most of these wind wheels is about 0.55 of the unloaded speed, and a variation of speed between 0.45 and 0.70 of the unloaded speed will not make a very great difference in the amount of work performed, provided the load is varied to suit the difference in speed.

BEST LOADS FOR WIND WHEELS.

In the eighth column of the preceding table the decimal fractions $\frac{C}{D}$ indicate the ratios of loads at the maximum of work to the greatest loads that the wheels can start without continuous turning.

In collecting from the original tables the numbers *D*, which represent starting forces, the original figures were corrected so as to exclude what was originally added for axial friction. (See "Starting forces" *on page 25.)* Where 0.3 pound was added to applied loads prior to *March 13, 1883,* the figures were corrected for C in the sixth column

WINDMILL AND RESERVOIR

correspond to an axial friction of 0.15 pound, as explained under the ading "Corrections for axial friction," page 75.

In general, it may be said that the ratio of best loads to starting rces is about 0.55, and that there may be a variation in the load ıtween 0.50 and 0.65 of the starting force without serious impairment efficiency.

CONICAL DEFLECTOR IN CENTER OF WHEELS.

Wheel No. 50 was tested because our attention was called to a wind-ill patent in which a conical deflector was shown. Besides saving ıe wind that otherwise would pass through the center of wheel noccupied by sails, an advantage was also claimed because the air as made to act with greater leverage on account of encountering the ıils farther from the axis. The experiment showed there could be no ossible advantage from such an arrangement.

AIR-CUSHIONED SAIL SURFACES.

When wheel No. 39 was constructed, the narrow strips of board long the edges of the sails were intended not to obstruct the spaces etween sails, but to afford air cushions on the faces of the sails against hich the wind might act to better advantage than against plain sur-ces. It had become apparent that plain sails were decidedly inferior sails with concave faces, whether circular or angular. (Note experi-ents with wheels Nos. 40, 43, and 44.)

It was contended by certain windmill experts that the narrow strips ded to plain sails, as used in wheel No. 39, would answer the same rpose as the sail forms of wheels Nos. 40, 43, and 44, and that wooden ıeels could be more easily constructed in that way. One of the perts had constructed wind wheels similar to wheel 39. It was mediately due to the suggestion of another prominent windmill offi-ıl that wheel No. 39 was made and tested. For ourselves, we had ıch more decided opinions after the test than before, notwithstanding e fact that our mathematical solutions as presented in this paper were itten out and made public before any of our experiments were com-ınced. Whatever value there may have been in the air-cushioned il faces of wheel No. 39, the advantage was more than nullified by e obstruction of spaces between sails and the bad contours of their cks.

OBSTRUCTIONS ON THE BACK OF SAILS.

Our experience with wheel No. 39 suggested that the arms and bands so might be detrimental to the proper smooth and peaceful flow of air ıtween the sails. So we next constructed and tested wheels Nos. 45 49, which have no bands and no arms along the backs of the sails. he results, as shown in the table on page 59, speak for themselves.

It may be noticed that wheel No. 47 carried 3.3 pounds as best load, ıe same as the best *load of* wheel No. 2, over which it shows a gain *power of about 83 per cent.* The gain, therefore, in this case was

entirely in speed. Notice also the ratio between the speeds of the two wheels running unloaded, $\frac{148.68}{77.85}=1.91$. The denominator, 77.85, is taken from the table on page 27.

WASTE OF POWER DUE TO THE ARMS OF WIND WHEELS.

Two or three months after making the many experiments the summary of which is recorded on page 59, it occurred to us that it would be interesting to know and easy to ascertain something definite as to the gain in power effected by the removal of those portions of the arms which usually are made to extend along the backs of sails or between the sails of wind wheels. To represent the portions of arms previously removed, we prepared six pieces of pine wood 1 by 1¼ by 17 inches. These could be quickly attached to the backs of the pasteboard sails by two little screws in each, and as quickly removed, as often as necessary.

We found, on retesting wheel No. 48 in September, that it showed a gain over wheel No. 2 of 84 per cent instead of the 87 per cent gained when tested in June. We attributed this small loss to the drying and warping of the sails, and therefore designated the retested wheel as No. 60. The same wheel, with the six pine strips above mentioned fastened to the backs of the six sails, was designated as wheel No. 61. We found by repeated trials, as usual, that wheel No. 60 required, for the wind velocity taken to produce a maximum product, a load of 1.9 pounds.

This load was the same as the best load for wheel No. 2. The coincidence of best loads made the comparison of wheels No. 60 and No. 2 merely a matter of recording speeds. No time had to be consumed in changing loads and readjustments of the brake, and the two wheels were replaced by each other alternately in rapid succession.

The same method was pursued in comparing wheel No. 60 with wheel No. 61, except that in this case, instead of removing from the axle and replacing the entire wheels, we merely placed and removed in succession the six strips of pine. The table on page 61 gives the averages of results.

It is possible that 1.9 pounds may not have been exactly the best load for wheel No. 61. To facilitate the speed comparisons and render mistakes of brake adjustment impossible, the load was not changed and the adjustment of the brake was left undisturbed during these alternate tests of the two wheels. We consider it altogether probable that an exact determination and application of the best load for wheel No. 61 would not have materially changed the product as recorded. At any rate, simply removing the six strips of pine from the backs of the sails caused an increase in the speed of the wheel with the same load, from 38.13 to 56.50 turns per minute, or 48 per cent.

TWISTED SAILS.

at the angle of weather of a sail should be greatest at the end ›st the axle and least at its outer end where its speed is compara- ; great, seems self-evident, and the advantages to be gained by ing or twisting the sails so as to make the weather angles increase ıey approach the center have always been recognized, at least in y, by windmill manufacturers, although comparatively few Ameri- rind wheels are made with such sails. Our best results, as shown e table on page 59, were obtained with warped sails, but it does ıppear from previous experiments that the warping was by any ıs the most important feature of our best wheels.

e tables on pages 51 and 53 afford, in the comparisons of wheels 34 and 43, having twisted sails, with wheels as nearly like them ossible without twisted sails, an apparent test of the advantages ›d by making the weather angles greater at the inner ends of the Judged by these two wheels the advantage is very slight.

e are now satisfied that merely twisting the sails can not do much for the reason that what is gained in this way by better disposi- of the forward or receiving edges of the sails is largely lost by the ›quent less advantageous disposition of the rear edges from which rind escapes.

we have pointed out in the discussion of the theory of the wind- the rear edge of a sail should be nearly parallel to the plane of ›n without regard to radial distance, while the forward edge should arallel to the relative direction of the impinging current of air, h depends on radial distance.

is theoretical disposition would make the angles of weather great- here the sail speed is least, but is a very different thing from the ing of the sails, as in wheels Nos. 34 and 43.

ɡ. 26, on page 61, shows a sail form more nearly in accord with heory, which makes not only the angle of weather, but also the ıture of the sail, greater with nearer approach to the center of heel.

DEFLECTORS IN FRONT OF SAILS.

our discussion of the theory of the windmill on pages 71 and 72 we called attention to the fact that our formula (E) indicates that the ency of a wind wheel might be increased if the air currents, just e meeting the sails, should be deflected so as to meet the plane of ›n at a less angle than 90 degrees. In order to test somewhat this ıematical suggestion, the experiments recorded in the table on page ere made. Only a single deflector was used, and that a plane sur- It may be noticed that when placed in front of wheel No. 2 at vari- ıngles the deflector caused some increase of speed in every instance, e the *load remained the same.* But when placed in front of wheel *9 its effect as an obstruction* overbalanced its value as a deflector.

It did not appear that deflectors would be advantageous in front of the most efficient wheel, nor of sufficient value in connection with the wheel of relatively low efficiency to justify the expense of such cumbersome appliances. We did not, therefore, consider it worth while to experiment further with deflectors.

OBSTRUCTIONS IN FRONT AND IN THE REAR OF WIND WHEELS.

The experiments recorded on page 65 we were prompted to make on account of the contention by certain windmill experts, that an obstruction behind a wind wheel caused about the same loss of power as if placed in front.

Many so-called vaneless windmills had their supporting masts, or tower tops, in front of the wheels, while the ordinary mills with vanes had their supporting masts behind the wheels.

The obstructing boards described under the table on page 65 were intended to represent such obstructions as would be representative of supporting masts placed as indicated. In the case of wheel No. 2, unloaded, the obstruction when placed in front caused a loss of speed amounting to a little more than 3.5 per cent, and when placed behind the loss amounted to a little over 3 per cent.

In the case of wheel No. 49, loaded, the loss of speed from placing the obstruction in front amounted to nearly 14 per cent, and the loss of speed from placing the obstruction behind was nearly 7 per cent.

It may be observed that 3.5 pounds is less than the best load for wheel No. 49 (3.9 pounds), but it should be borne in mind that with all wheels a considerable variation in the load made but little difference with the product when near the maximum.

It might have been profitable if we had extended these experiments. It is evident that obstructions even when relegated to the rear are not obliterated.

MAXIMUM PRODUCTS IN DIFFERENT VELOCITIES OF WIND.

The table on page 66 indicates that maximum products vary as the cubes of the velocities of wind almost as exactly in fact as in theory. If there had been an exact correspondence between the results of experiment and the law of cubes, the figures under $\left(\frac{b}{a}\right)^3$ would exactly equal those opposite under $\frac{B}{A}$. The figures under $\frac{B}{A}$ are in some instances a little greater and sometimes a little less than those opposite under $\left(\frac{b}{a}\right)^3$, as indicated by the plus and minus signs before the differences given under $\frac{B}{A} - \left(\frac{b}{a}\right)^3$. The plus and minus signs happen to be equal in number, showing a remarkable general agreement with *the law of cubes.* Again, the law of cubes would require that the

numbers under A multiplied by $\left(\frac{b}{a}\right)^3$ should equal the corresponding numbers under B. How nearly the law was actually fulfilled may be seen by comparing the last computed column, $A \times \left(\frac{b}{a}\right)^3$, with the real products of column B.

STARTING FORCES AND LOADS AT THE MAXIMUM IN DIFFERENT VELOCITIES OF WIND.

Under the second table on page 66 we find the conclusion that "starting forces vary as the squares of the velocities of wind." If the law of squares had been strictly confirmed, the computed ratios under $\left(\frac{b}{a}\right)^2$ would have just equaled the actual ratios of experiment as given under $\frac{B}{A}$. The agreement is quite close, but the numbers in column $\frac{B}{A}$ are more often greater than the corresponding numbers in column $\left(\frac{b}{a}\right)^2$. So it appears that starting forces really increase in at least as great a ratio as would conform to the law of squares. But, as we have explained before, it should be remembered that what we have called starting forces could neither be defined nor determined with the same accuracy as was attainable in the case of maximum products. Nor could the loads corresponding to maximum products be determined so accurately as the maximum products themselves, since two or three different loads would generally give about the same product when the product was near the maximum. This fact made it all the more easy to ascertain the maximum products correctly, since a little variation from the best load was compensated for by the consequent variation in the speed of the wheel.

The table on page 77 shows, on the whole, a fairly constant ratio between loads at the maximum and starting forces, from which we may draw the conclusion that loads at the maximum vary as the squares of the velocities of wind.

SPEEDS OF UNLOADED WHEELS IN DIFFERENT VELOCITIES OF WIND.

In the table on page 67 an inspection of columns $\frac{b}{a}$ and $\frac{B}{A}$ will show that in every case but two the counted turns of the unloaded wheel in the higher wind exceeds somewhat the number of turns computed on the supposition that the speed of an unloaded wheel should vary directly as the velocity of wind. The two exceptional cases may be accounted for. We attribute this variation from the natural law in part to the fact, previously discussed, that the axial friction diminishes as the speed increases. *Strictly speaking*, the wheels, on account of unavoidable *friction at the axle, could* not be entirely unloaded.

ACTION OF AIR ON THE SAILS OF A WIND WHEEL.

In fig. 31, AB, AB are supposed to represent cross sections of two sails, one following the other as in a wind wheel. DE represents the direction and velocity of the wind with respect to the earth. AE represents the direction and velocity of the sail sections, at right angles to DE. Then DA represents the relative direction of wind with respect to the sails in motion, and the front faces AA of the sails are set parallel to DA. The rear faces of the sails B are set parallel to the plane of the wheel's motion, according to correct theory.

As one sail necessarily follows another in a wind wheel, the air can

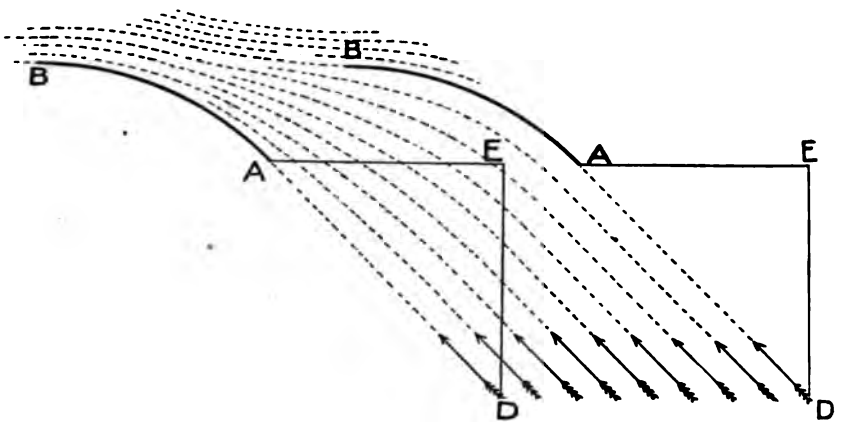


FIG 31.—Action of air on the sails of a wind wheel.

not escape in a direction exactly opposite to the sail's motion, and as the space between sails unavoidably contracts toward the rear edges these edges may in practice conform in some measure to the best possible direction of escape instead of being strictly parallel to the plane of the wheel. The placing of arms on the backs of or between the sails would render a necessary evil worse on account of further contraction of space. Besides, it should be noted that the backs of the sails, as well as their faces, play an important part in deflecting the wind. For it is a well-known fact that air currents cling to and follow the surfaces along which they flow. Therefore, the backs of the sails should be smooth and free from obstructions. It is evident that all the air passing between the sails is deflected and not merely that portion which comes immediately in contact with the faces of the sails. The arrows and dotted lines in the figure indicate the relative direction and deflection of air as it flows through the wheel.

THEORETICAL EFFICIENCY OF THE WIND WHEEL.

Our general formulæ and deductions previously given under theory *of the windmill* are presented just as they were originally recorded *near the beginning of* the year 1882. We now see no reason for alter-

ing them in any respect, but there has been so much confusion among mathematical writers on wind action, that we wish to make the matter perfectly clear in its essential points.

It is not necessary to resort to the calculus, nor even to produce the general formulæ (A) and (B) in order to arrive at the theoretical efficiency of a wind wheel. A very simple, graphical solution may be shown as follows: In figs. 32 to 35, AB represents the direction and velocity of the wind with respect to the earth; AC represents the direction and velocity of the sail, a section of which is represented by the curved line CD; the velocity of wind AB is the same for each of the four cases; and the several sail sections are supposed to be taken at the extremity of a sail in fig. 32, at a distance from the center of the wheel equal to two-thirds the radius in fig. 33, at one-third the radial distance in fig. 34, and at one-sixth in fig. 35. We have assumed that

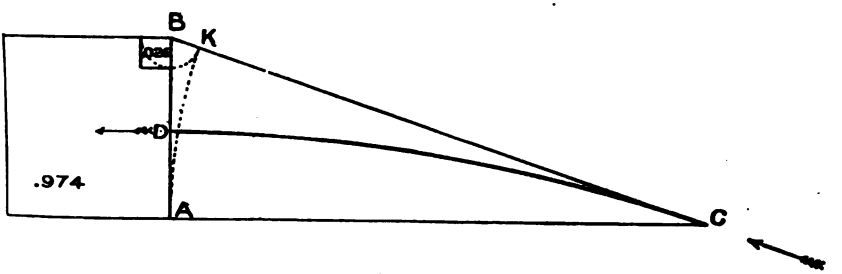


Fig. 32.

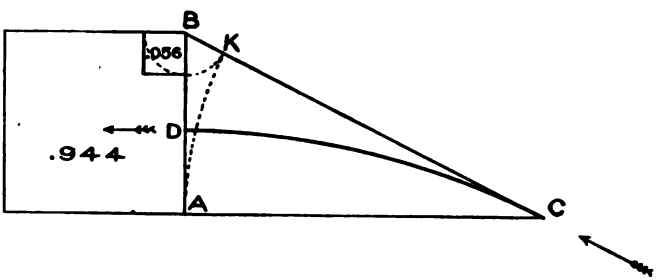


Fig. 33.

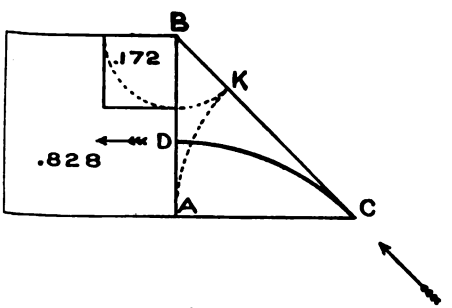


Fig. 34.

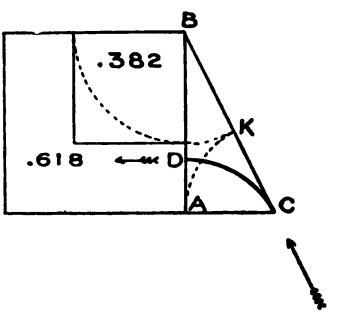


Fig. 35.

FIGS. 32–35.—Theoretical efficiency of the windmill.

the outer extremity of the sail travels three times as fast as the wind. That is to say,

$$AC = 3 \times AB \text{ in fig. } 32,$$

$$AC = 2 \times AB \text{ in fig. } 33,$$

$$AC = 1 \times AB \text{ in fig. } 34,$$

$$AC = \tfrac{1}{2} \times AB \text{ in fig. } 35.$$

CB represents the relative direction and velocity of the wind with respect to the sail in each case. Neglecting friction, the relative velocity of the wind is not changed by contact with the sail. The air escapes from the sail at D with the same relative velocity it had at C; but its course is changed so that it escapes in a direction parallel and opposite to that of the sail's motion. Now, since the sail is traveling with an actual velocity AC in one direction while the air is escaping with a relative velocity CB in the opposite direction, the difference between CB and AC = BK will represent the actual velocity of the air after it has escaped from the sail. The remnant of velocity BK is all there is left of the wind's original motion. The sail has absorbed the rest and converted into work a corresponding amount of energy. A certain quantity of wind started with a velocity AB and gave up all its energy except that corresponding to a velocity BK.

Now, the potential energy of the same quantity of air in motion varies as the square of its velocity, as in the case of any other substance. Hence a square constructed on AB and another square constructed on BK will represent relatively the original potential energy of the wind and the energy it still retains after encountering the sail. These squares are shown in the figures. The small squares represent the waste of energy or that not consumed by the sail. The relative areas of these squares may be readily computed. Taking the large squares as unity the small square equals 0.026 in fig. 32, 0.056 in fig. 33, 0.172 in fig. 34, and 0.382 in fig. 35. The efficiency is represented by the difference between the squares; thus, the efficiency in fig. 32 is 0.974; in fig. 33, 0.944; in fig. 34, 0.828, and in fig. 35, 0.618.

The small squares correspond to the letter k in the formula (B) on page 71. Compare these efficiencies with the values of $1-k$ as originally obtained by a very different process. There would have been no difference if the decimals had been equally carried out in both cases.

The potential energy of the wind, as well as the useful work resulting from its action on a sail, varies as the cube of its velocity, because the quantity of air coming in contact with the sail increases or diminishes with the change of velocity in the same proportion. It is only when we consider the result of a fixed quantity of air in motion, that *its energy varies as the* square of velocity and may be represented by *square areas as illustrated* above.

e work of the same quantity of air varies as the square of velocity, ıse its resultant reaction against the sail varies directly as the ity and the sail also travels with the same proportional velocity. r instance, 1 pound of air, acting on a sail, has its velocity doubled, ıail travels with double velocity under doubled pressure, so that vork is increased fourfold; and, as in reality the doubling of veloc- ould cause 2 pounds of natural wind to meet the sail in the same the result is doubled again, making it eightfold in theory as well practice.

THEORETICAL USEFUL EFFORT OF WIND.

the figs. 36 to 39, let AB represent the direction and velocity of with respect to the earth. In fig. 36 let AC = 3 AB represent. direction and velocity of the outer extremity of a wind wheel; in 7 at two-thirds of the radius from center, AC = 2 AB; in fig. 38 ıe-third the radius from center, AC = AB, and in fig. 39 at one- ı the radius, AC = ½ AB. The circular arc CD in each figure ›sents correctly what the cross section of the sail at each point ld be according to theory. To make the calculation simple and fic, we will consider the action of 1 pound of air only, and that ng at an original velocity of 32.2 feet per second.

e take this particular velocity because it requires the constant ›n of a 1-pound force to give to 1 pound of air, or to 1 pound ıy substance, a velocity of 32.2 feet per second.

› give other velocities to the 1 pound of air, requires the impelling ›n of a force proportional to the velocity imparted in a second of . That is to say, the force required to impart to 1 pound of air velocity, v, is represented in pounds by $\frac{v}{32.2}$.

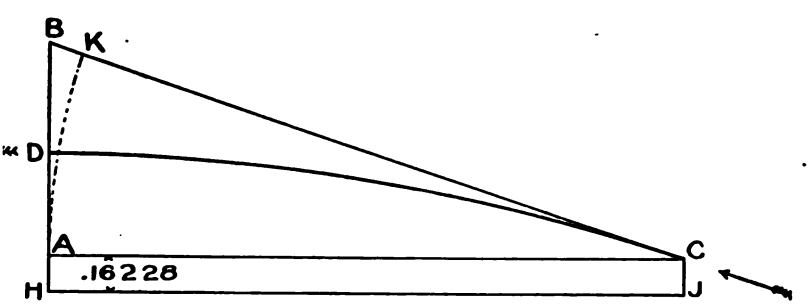


Fig. 36.

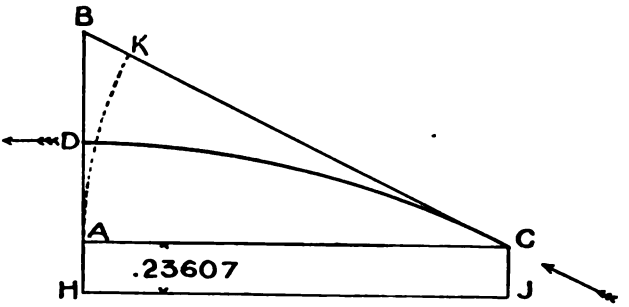


Fig. 37.

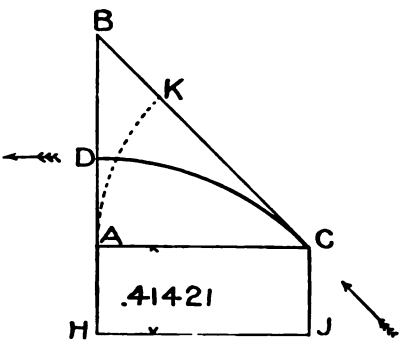


Fig. 38.

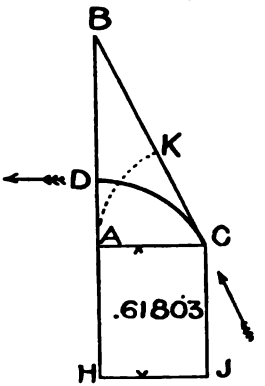


Fig. 39.

FIGS. 36-39.—Theoretical useful effort of wind.

The sail of a wind wheel moves at right angles to the direction of wind or in the direction AC, but the wind meets the sail in motion in the relative direction and with the relative velocity CB, and has with reference to the sail, due to the sail's motion, an opposite motion equal to CA in the direction C to A in addition to its original motion. The length of the line CB represents the velocity with which the air flows along the surface of the sail with undiminished relative velocity, neglecting friction, and the difference between CB and CA, or BK, represents the relative velocity acquired in a second by the air in a direction parallel to CA. That is to say, by action against the sail during one second, the 1 pound of air has acquired a relative velocity BK in a direction parallel to CA, in addition to the relative velocity CA which it had before meeting the sail. This newly acquired relative velocity BK required for its generation a force reacting between the sail and the air proportional to the acceleration of velocity. If BK were equal to 32.2 feet, the reaction would be 1 pound. But the reaction is less than 1 pound in the same ratio that BK is less than AB, which we assumed to be 32.2 feet.

AB:BK:: 1 pound: Useful effort of wind.

So $\frac{BK}{AB}$ *represents* the ratio between original wind pressure in the

WINDMILL AND RESERVOIR.

direction of its original motion and the useful effort which pushes the sail in the direction of its motion. If we lay off on BA prolonged AH=BK and complete the rectangles ACJH, the areas of the four rectangles in figs. 36 to 39, will represent relatively the work theoretically performed in each case. The effort AH may be readily computed. For the action of 1 pound of air, at velocity of 32.2 feet per second, we find that the useful effort is 0.162 pound when the sail moves 3 times as fast as the wind, 0.236 pound when the sail moves twice as fast as the wind, 0.414 pound when the sail moves as fast as the wind, and 0.618 pound when the sail moves one-half as fast as the wind.

AERIAL RESISTANCE TO MOTION.

It should be noted that by far the most striking and important results of our experiments with wind wheels were produced by cutting out from between the sails obstructions which produced aerial resistance to motion; and it should be profitable to study the causes which make these aerial obstructions apparently a matter of greater moment than all the other features of our investigation.

In the two preceding articles we have shown that the greater the sail velocities, theoretically the higher the efficiencies, and also that the useful efforts become smaller as the sail velocities increase.

When the sail travels three times as fast as the wind, the theoretical efficiency is 0.974, very close to perfection; but for a wind velocity of 32.2 feet per second, each pound of air gives a useful effort of only 0.162 pound. It is evident at once that while high sail velocity seems desirable, it does not require great aerial resistance to counteract a large percentage of the useful effort. When the sail travels three times as fast as the the wind, a flat resisting surface of about 1 square inch set perpendicular to the direction of motion would counteract entirely the useful effort due to the action of 1 pound of air. This is computed as follows:

At a velocity of 32.2 feet per second, 1 pound of air per second under ordinary conditions represents a stream of about 59 square inches area in cross section; from which it follows that 59 square inches of flat surface carried normally against still air at the rate of 32.2 feet per second meets with a resistance equal to 1 pound, and at a velocity three times as great meets with a resistance of 9 pounds. But the total useful effort in this case is only 0.162 pound.

Therefore we obtain $F=\frac{59\times 0.162}{9}=1.062$ square inches=flat resisting surface required completely to neutralize all useful effort of wind whose normal pressure against 59 square inches of flat surface equals 1 pound. In like manner we find that F equals 3.418 square inches where the sail velocity is twice the wind velocity, 24.426 square inches where the sail velocity equals the wind velocity, and 145.848 square inches *where the sail velocity is one-half* the wind velocity.

Or if we let f be the percentage of wind area represented by F, we find that $f=0.018$ when sail velocity is three times wind velocity, 0.059 when sail velocity is twice the wind velocity, 0.414 when sail velocity is equal to wind velocity, and 2.472 when sail velocity is one-half wind velocity.

These percentages hold good for all velocities of wind and emphasize the necessity of reducing to the utmost aerial resistances to motion, if we are to attain high efficiency in a wind wheel.

BEST NUMBER OF SAILS.

An inspection of the table on page 65 shows that wheel No. 35 was more than 3 per cent better than wheel No. 19, although the amounts of sail surface and the angles of weather were the same in both. Wheel No. 35 had exactly the same sails as No. 19, except that each sail of No. 35 contained two of No. 19 placed edge to edge, making 12 broad sails instead of 24 narrow ones. By simply changing the angle of weather from 27.5 degrees to 25 degrees, we made No. 35 into No. 36 and thus gained nearly 6 per cent over No. 19. The wide sails did better with less angle of weather. Wheel No. 6, with the same number of sails as No. 19 but narrower, required a greater angle of weather, 30 degrees.

It is apparent that for a given total of sail area it is better to divide the surface between fewer sails. It may be noted that wheel No. 48, with six sails, gave, in proportion to total sail area, nearly two and a half times the efficiency of No. 2 with 60 sails. Reducing the number of sails reduces the aerial resistance to motion due to the number of edges, and leaves relatively freer interstices for the flow of air between the sails.

BEST RELATIVE AREAS OF SAIL SURFACE.

Tables on pages 47 and 49 give the following results for wheels Nos. 19, 24, and 29:

Effect of area of sail surface on efficiency.

	No. 19.	No. 24.	No. 29.
Square feet of sail surface	12.937	15.000	17.016
Ratio of maximum products (*a*)	1.204	1.212	1.201

a Compared with wheel No. 2.

These three wheels were nearly of the same efficiency. As compared with the area of the zone containing the sails, No. 19 was about three-fourths full, No. 24 was about seven-eighths full, and No. 29 was what we call full, as the total area of the sails about equaled the area of the zone containing them. These wheels all contained the same number of sails set at the same angle, and differed only in width of the sails. It is clear that nothing was gained by making the total sail area more than seven-eighths of the zone, and that very little was

gained by filling beyond three fourths of the zone—surely not enough to pay for the extra material.

From the table on page 65 we obtain the relative values of these three wheels per square foot of sail surface, viz, for No. 19, 0.0931; for No. 24, 0.0808; and for No. 29, 0.0706. Notice also the relative values per square foot of sail surface as given for other wheels in this table.

ACTUAL EFFICIENCY OF THE WIND WHEEL.

To obtain the actual efficiency of wind wheel No. 2, taking the result recorded in the table on page 27, we first obtain the maximum work performed by multiplying the maximum product by the circumference of a circle whose diameter is 2 feet. (See note under the table on page 28.)

62.463 = maximum product in the table.

6.283 feet = circumference of circle 2 feet in diameter.

62.463 × 6.283 = 392.455 foot-pounds per minute = work of wind wheel No. 2 at maximum.

The potential energy of the wind is expressed by the well-known formula $\frac{W v^2}{2g} = \frac{W}{32.2} \times \frac{v^2}{2}$ in which we make W = weight of air intercepted per second by the total area of the wind wheel. v = velocity of wind in feet per second, and $g = 32.2$, the velocity acquired each second by a body whose motion is not resisted when impelled by a constant force equal to its own weight. The temperature as recorded was 50° F., and on the same day at 2 p. m., in Chicago, the barometer stood at 28.983. Hence 0.075 pound may be taken as the weight of 1 cubic foot of air, 19.635 square feet as the total area of the wheel which was 5 feet in diameter, and 12.286 feet per second, or 8.403 miles per hour, as the velocity of wind. Thus, $W = 19.635 \times 12.286 \times 0.075 = 18.093$ pounds per second, $v^2 = 150.946$, and $\frac{W v^2}{2g} = \frac{18.093 \times 150.946}{2 \times 32.2} = 42.4078$ foot-pounds per second, or 2544.468 foot-pounds per minute. The portion of actual energy of wind utilized by wind wheel No. 2, or its efficiency, is accordingly $\frac{392.455}{2544.468} = 0.154 +$.

To obtain the efficiency of any other wind wheel whose maximum product is directly compared with maximum product of wheel No. 2 as shown in the tables of later date than March 13, 1883, it is only necessary to multiply 0.154, the efficiency of wheel No. 2, by the ratio of products obtainable from the various tables.

For example, in the table on page 59 we find 1.872 given as the ratio of maximum products for wheels No. 48 and No. 2. Hence, 0.154 × 1.872 = 0.288 is the efficiency of wheel No. 48. This is the highest efficiency attained by any of the wheels whose products are recorded in the preceding tables.

WORK OF VARIABLE WIND WITH VARIABLE LOAD.

In most localities the velocity of the wind is exceedingly variable. The average of velocity recorded for an hour by an anemometer gives no idea of the total actual energy of the wind.

It might in conception be actually of uniform velocity, but where the average velocity is 10 miles per hour it is quite as likely to blow one-half the time at the rate of 20 miles and during the other half not at all. The potential energy of 20-mile wind for half an hour is four times as great as that of a uniform 10-mile wind for a whole hour, although the average velocity per hour in each case is the same.

This is taking an extreme case, but it shows how the wind as it blows may and does develop much more energy than could be derived from uniform wind of equal velocity as usually recorded. It would be desirable, if possible, to have the load applied to a wind wheel vary as the square of the velocity of wind, in order that the work might be the greatest possible. Any fixed load which allows the wheel to run freely most of the time is generally altogether too small during much of the time as well as too great at other times. For most uses to which wind wheels are especially adapted an automatic regulation of the load to meet the varying wind would make the best form of wind wheel regulation.

REGULATION OF WIND WHEELS.

Ordinarily wind wheels in this country are made to regulate themselves automatically, so that they can not attain a very rapid rate of rotation. This practice of not allowing the wheel to run at more than a very moderate speed is due to the fact that our wind wheels have been developed mostly in connection with the operation of common reciprocating pumps attached directly to a crank on the shaft of the wheel. Such direct connection necessitates a slow motion of the wheel, in order that the piston speed of the pump may not exceed its economical limit. The consequence is that many wind wheels are allowed to do only a small part of the work which they are capable of developing. Too much attention has been paid to restraining automatically the speed of wind wheels by means which involve a waste of power. So far as the safety of the wheel itself is concerned there should be no need of restraining its speed in any wind under 40 miles per hour; and it should be borne in mind that strains on the wheel, due to any given amount of work, may be diminished in the same ratio that the speed of the wheel is increased.

The gearing of a wind wheel, whether it is geared up or down, should be made to suit the work and the nature of the device through which the power of the wheel is applied to use. When the machine driven requires a high speed of revolution, much gearing up may be avoided if the wind wheel is allowed to revolve rapidly. Back gearing for slow running machines should be resorted to whenever a direct connection

ith the machine to be driven will not allow the driven mechanism nd the wind wheel both to have their best speeds with reference to ork and wind.

For any given velocity of wind the speed of the wind wheel should ot change, no matter what the nature of the work or of the machine hrough which work is performed; but the gearing should be such as suit the load to the wind.

LIMITATION TO SIZE OF WIND WHEELS ON ACCOUNT OF WEIGHT.

Probably in no other form of motor does a gain in efficiency effect so reat a saving in weight as in the wind wheel. The strength of other notors needs to be proportional to their own power. A 4-horsepower team engine for instance does not need to weigh more than four imes as much as a 1-horsepower engine, as the strains to which it s subjected are only four times as great. But a 4-horsepower wind wheel needs to be eight times as heavy as a 1-horsepower wheel in order to resist equally well the violence of exceptional storms. It is he strains of storms, and not its own working strains, that determine he required strength of a wind wheel.

Consider a single sail A supported on the end of an arm fastened to a central spider. Let L = length of the arm from center of wind pressure on sail to point of attachment to spider. If the wind wheel is loubled in diameter, keeping the same proportions, L will be doubled and the area of A will be four times as great. Hence the arm needs to resist four times the wind pressure acting at double the former distance out from its point of attachment, and needs to have eight times its former strength. This eightfold strength is secured by doubling all lineal dimensions, as evidently should be done, when the diameter of a wind wheel is doubled.

But the doubling of all lineal dimensions makes the wheel weigh eight times as much, and as its area is only four times as great, it follows that, in proportion to its power, the weight of the wheel is doubled.

As actually made, all the dimensions of large wind mills do not conform strictly to the proportions of small mills, and often the large wheels are built after different models from small ones. But it is well-known that the large wheels as built do not resist storms so well as small ones, and costly experience has taught manufacturers to make the weight of their large wheels at least approximately what calculation requires.

On account of the disproportionate weight of large wind wheels, the towers which support them also have to be made considerably heavier than would be the case if the large wheels required no more material in proportion to power than do small wheels. If the tower for a large wind wheel were of correspondingly greater height, the weights of towers would follow the same law that should control the weights of wheels. It is evident that small wind wheels are more efficient in proportion

to weight than large ones, and that the cost of construction, as the wheels are made larger, is increased in much greater proportion than the gain in power.

EFFICIENCY OF WIND WHEELS AS AFFECTED BY DIAMETER.

Another reason why small wind wheels are more efficient than large ones is that the wind meets all parts of the area of a small wheel with greater uniformity of velocity. It needs no special acuteness of observation to discover that the wind strikes different portions of large wheels with great unevenness of velocity.

Before commencing the experiments here recorded we attempted to measure the power of a wind wheel 22 feet in diameter in natural wind. By way of preparation we made a special anemometer, which indicated at sight the velocity of wind. We placed the anemometer as near to the wind wheel as possible, expecting to be able to note where its pointer stood and know the wind velocity for a minute of time, during which the revolutions of the wheel carrying a known load might be counted. The first thing we learned was that the anemometer never pointed steadily to any uniform velocity of wind even for one-fourth of a minute, and we next observed that the anemometer would sometimes almost stop running when the wind wheel showed an extra spurt of speed; also the wind wheel would slow down while the speed of the anemometer was accelerated. We could often hear the wind whistling through the sails on the opposite side of the wind wheel, while very little wind was felt on the near side close to the anemometer. We were unable to obtain a single measurement which we considered worth preservation. We could not determine even the best load for any wind, to say nothing about comparative results. We do not doubt that localities might be found where the wind would blow with greater uniformity, but our experience in trying to measure the power of wind wheels in natural wind led us to provide for artificial wind before proceeding further. The slight variations in the best artificial wind we could command caused a great abundance of vexations, which made the obtaining of accurate average results a matter of tedious labor.

HEIGHT OF TOWERS.

Nothing like steady wind can be obtained near the ground in an inhabited country. Buildings, trees, and other obstructions set the wind to whirling and cause it to flow in sinuous streams of very uneven velocity, concentrated in one place at the expense of another.

There is no effective remedy except in the elevation of the wind wheels several feet at least above all obstructions, even if the obstructions are isolated and a thousand feet away. In order that a large wind wheel may be as efficient in proportion to its area as a small wheel, its height above the ground must be greater. So it is probably not far out of the way to say that the weights of the towers as well as

WINDMILL ON LOW TOWER ON THE GREAT PLAINS.

ie weights of the wind wheels, for equal safety and efficiency, should nearly proportional to the cubes of the diameters of the wheels.

MULTIPLICATION OF WIND POWER.

We have shown that for equal safety in storms the weights of wind heels of different sizes and like forms should be proportioned to the ibes of their diameters. It would require four 12-foot wheels to equal e area and power of one 24-foot wheel, if the larger wheel is propor- nately elevated. But the weight of the one 24-foot wheel would be ice as great as the combined weight of the four 12-foot wheels, and e weight of the one higher tower would probably be twice that of e four shorter towers combined. Hence it would seem that in pro- rtion to the power obtained in each case, the one 24-foot wheel would st twice as much in material. The thought naturally presents itself at the four 12-foot wheels ought in some way to be combined so as to t in unison for concentrating a great amount of power where it is sirable to use the power at only one point, as in driving one machine large dimensions. If the four wheels were coupled together rigidly e trouble from uneven reception of wind which is experienced in ge wheels would be augmented. The problem has not been worked t, but we may imagine a number of wind wheels, each compressing according to its own ability and delivering it at any distance into a nmon reservoir. Natural elevations would be selected as locations windmills, and such a plant could not be rendered useless for the ie by an accident to one or two of the wind wheels. There would essarily be considerable loss in compressing air, but a low-pressure stem might be devised that would greatly reduce the waste. Some ste of power attends every mode of transmission. In seeking to ke a gain in power of 100 per cent in proportion to cost of plant, loss of an extra 25 per cent in transmission might well be tolerated. There may, however, be other and better methods for accomplishing object in view than by the means we have ventured to suggest.

POWER OF TWELVE-FOOT WIND WHEEL.

The maximum product of our best experimental wheel, No. 48, is given 247.123 in the table on page 59, and the velocity of wind as 10.935 les per hour.

The constant 6.283 multiplied by products gives the foot-pounds of rk per minute. Hence $247.123 \times 6.283 = 1552.674$ foot-pounds = work r minute of No. 48 in wind of 10.935 miles per hour.

The work of 5-mile wind compared with the work of 10.935-mile wind $\left(\frac{5}{10.935}\right)^3$ or .0956. Hence, the work of No. 48 is, in 5-mile wind, 52.674 × .0956 or 148.345 foot-pounds. The work of a 12-foot wheel 5-mile wind is $148.345 \times \left(\frac{12}{5}\right)^2$ or 854.467 foot-pounds, which is equal .0259 horsepower.

It is, therefore, approximately correct to call the power of a 12-foot wind wheel in 5-mile wind equal to one-fortieth of a horsepower. Computing the power of other wind velocities according to the law of cubes, we obtain for the power of a 12-foot wheel, one-fortieth or 0.025 horsepower in 5-mile wind, 0.2 horsepower in 10-mile wind, 0.675 horsepower in 15-mile wind, 1.6 horsepower in 20-mile wind, 3.125 horsepower in 25-mile wind, 5.4 horsepower in 30-mile wind, 8.575 horsepower in 35-mile wind, and 12.8 horsepower in 40-mile wind.

Most 12-foot wind wheels, as now made, are strong enough to stand the strain of furnishing 13-horsepower, and this we believe is not beyond the actual achievement of some 12-foot wind wheels in 40-mile wind.

VALUE OF MATHEMATICAL FORMULÆ IN EXPERIMENTAL INVESTIGATIONS.

The formulæ which Professor Rankine and other mathematicians have applied to wind wheels give for maximum theoretical efficiency only 50 per cent of the potential energy of the wind actually intercepted by the sails. Our experimental wind wheel No. 48 realized an efficiency of more than 44 per cent, if we consider only the wind which would be intercepted by a surface equal to the projections of the sails on the plane of the wheel. So, if we should accept previous mathematical calculations as correct, we have already made a wind wheel which utilizes 88 per cent of the greatest possible theoretical result, and the remaining 12 per cent margin would not furnish very great incentive to attempts at improvement. Mathematicians hitherto have treated the sails of wind wheels as flat surfaces. Even for flat surfaces their theoretical results have been entirely too small; but it is not our purpose to discuss previous mathematical errors. Wind-wheel sails of proper form are not flat, but curved surfaces, and in analyzing the action of wind on curved sails we have not contradicted Rankine's formulæ, but have confined ourselves to making such suppositions as may be actually true in practice. Rankine's formulæ for the action of fluids on curved vanes give as a theoretical result a maximum efficiency of 100 per cent. This is correct. It shows that we have not made a very efficient wind wheel yet, and affords hope for the attainment of still better results in the future. We know that there is a great waste of energy somewhere, even in the best wind wheels we have been able to produce, but we are encouraged to think that much of the waste can be avoided. We do not claim, therefore, that our experiments have revealed the whole truth. In fact we believe that only a beginning has been made, and fully expect to see our best results greatly surpassed. In our view, the results of our experiments, as here recorded, are far richer in the suggestions offered than in the efficiencies realized.

INDEX.

Page.

ance of..... 71, 76, 80, 89
ts, action of..... 84
ned sails, description of..... 57, 79
wind wheels, development of.... 19, 30, 74, 75, 92
er, description of..... 94
npingement, theory of..... 70, 71, 72
reather, experiments with.... 22, 75, 90
y of..... 72
c influences, effect of..... 27, 72, 76
tomatic adjustment of..... 27, 67, 68, 69
ption of..... 23, 24, 67
experiments with..... 46, 64, 79, 81
y of..... 71
eter, description of..... 23
of windmills, determination of... 22, 60, 91, 96
of area of sails on..... 90–91
of diameter of wheel on..... 94
y of..... 69–72, 84–87
wind. (*See* Wind.)
nts with windmills, methods of... 24, 25, 26, 27, 42
of..... 9, 21, 22, 72, 73
orrections for..... 25, 26, 38, 75–76
rake. (*See* Brake.)
f windmills, method of..... 92–93
friction of..... 25–26
windmills, application of..... 23–24
friction included in..... 24, 75
mination of best..... 77–79, 83
f squares for..... 66, 83
ation of variable..... 92
H., introduction by..... 11–18
of transmittal by..... 9
ng edges, effect of..... 90
ons before and behind wheels. 65, 82
ons on sails, effect of 57, 61, 62, 79, 89
wind wheel, determination of 95
determination of maximum..... 24, 27, 42, 73, 76, 82–83
of journal friction on..... 20, 22, 24, 25, 26, 75, 76
of state of atmosphere on.... 27, 72, 76

Page.

Regulation of wind wheels, methods of..... 92–93
Results of experiments, variations in 73
Sail area, determination of best..... 65, 90–91
Sails, angle of weather of..... 22, 41, 42, 51, 52, 90
Effect of twisting of 51, 53, 57, 81
Efficiency of..... 85
Forms of..... 65, 69, 72, 84, 85, 90
Inclination of. (*See* Angle of weather.)
Number of..... 28, 29, 65, 90
Obstructions on. (*See* Obstructions.)
Speed of..... 59, 71, 89, 93
Smeaton, John, experiments of..... 19, 20, 21
Speed indicators, forms of 23, 24, 42
Speed of wind wheels, determination of.... 25, 67, 77, 78, 83, 93
Starting forces, best angles for..... 40, 41
Definition and measurement of. 25, 66, 77–79, 83
Thermometric influences, effect of..... 27, 72, 76
Towers, height and weight of..... 93–94
Wind, effect of circular motion on 20, 21, 23
Efficiency and waste of..... 85
Energy of..... 69, 86, 91, 92, 95
Escaping currents of..... 69, 71, 72
Experiments in natural..... 92, 94
Impinging currents of..... 60, 70, 72
Theory of the action of 69, 72, 84
Use of artificial..... 21, 22, 42, 74
Useful effort of..... 87–89
Wind wheels, determination of power of .. 95
Determination of speed of...25, 67, 77, 78, 83, 93
Dimensions of..... 93–94
Effect of obstructions on..... 28, 29, 60, 64, 80
Efficiency of. (*See* Efficiency.)
Forms of..... 19, 20, 30, 74, 75
Methods of regulation of..... 92–93
Reversal of 34, 43
Sail area for..... 90
Size of 93, 94
Towers for..... 93–95
Work of wind wheels, computation of... 24, 91, 95
Graphic representations of..... 87, 88
In variable winds..... 92
Law of cubes for 66, 82, 86
Strains due to..... 92
Theoretical formula for 69, 70

IRR 20——7

UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

LLS OF NORTHERN INDIANA

BY

FRANK LEVERETT

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

CONTENTS.

	Page.
transmittal	7
tatement	9
liscussion of wells	13
County	13
r County	15
rte County	18
seph County	21
rt County	23
nge County	25
en County	27
lb County	29
County	31
usko County	35
hall County	37
e County	39
on County	40
r County	44
ki County	46
n County	46
ley County	47
County	48
s County	51
County	52
ington County	54
sh County	55
i County	57
County	59
e County	60
n County	61
en County	66
canoe County	67
ll County	70
on County	71
rd County	73
n County	75
t County	76
rford County	77
County	78
	81

ILLUSTRATIONS.

PLATE I. Map of Indiana showing the relation of the drift to the ordinary wells 9
II. Map of Indiana showing the Pleistocene deposits 12

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, October 20, 1898.

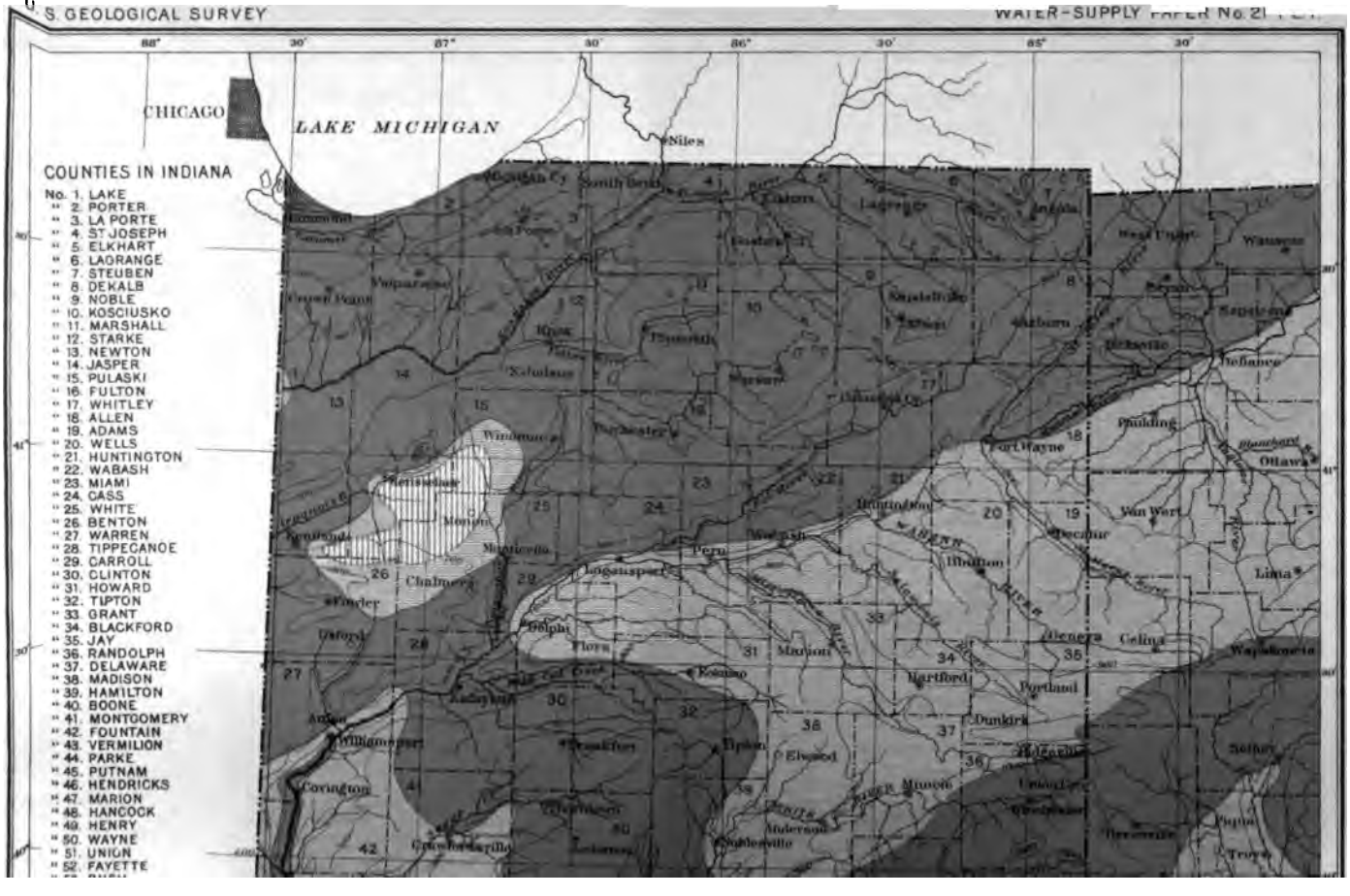
SIR: I have the honor to transmit herewith a report on the wells of diana, prepared by Mr. Frank Leverett, assistant geologist of this rvey. This material was brought together by Mr. Leverett in con-ction with glacial investigations, as noted in the general discussion the water resources of Indiana and Ohio, published in Part IV of › Eighteenth Annual Report of the Survey, on pages 419–559. ıny details were omitted in that publication, as the volume assumed lky dimensions; but these detailed facts have considerable value d are needed for reference by citizens of Indiana, and it is there-e desirable to make them available by publishing them in the series Water-Supply and Irrigation Papers.

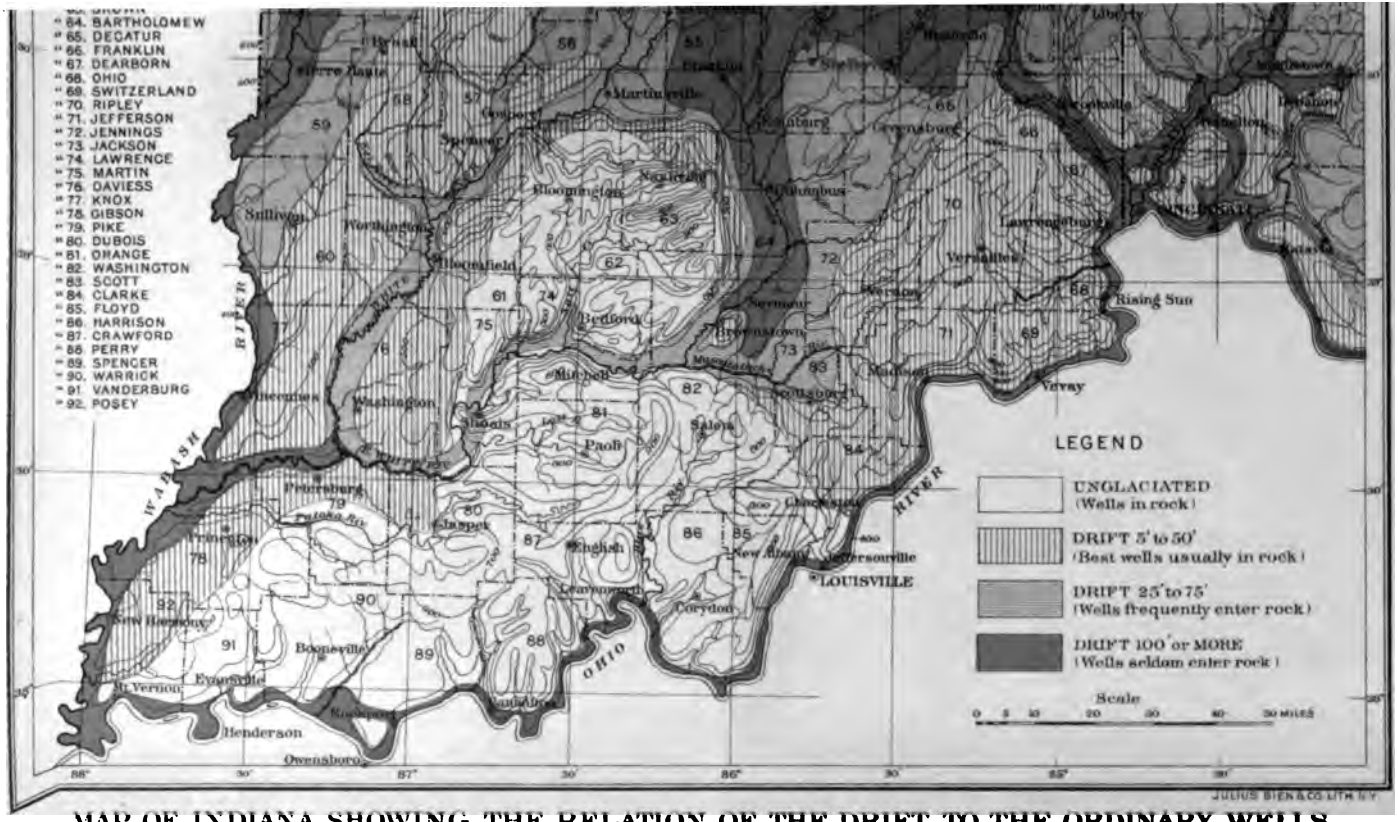
With these data are presented two maps, one illustrating the dis-bution of the glacial deposits and the other the relation of the drift the ordinary wells of the State. These are portions of Plates XXVI and XXXVII of Part IV of the Eighteenth Annual Report. eir reproduction in connection with this paper is essential to the ar understanding of the descriptive matter.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Ion. CHARLES D. WALCOTT,
Director United States Geological Survey.


CHICAGO
LAKE MICHIGAN
COUNTIES IN INDIANA
No. 1. LAKE
" 2. PORTER
" 3. LA PORTE
" 4. ST JOSEPH
" 5. ELKHART
" 6. LAGRANGE
" 7. STEUBEN
" 8. DEKALB
" 9. NOBLE
" 10. KOSCIUSKO
" 11. MARSHALL
" 12. STARKE
" 13. NEWTON
" 14. JASPER
" 15. PULASKI
" 16. FULTON
" 17. WHITLEY
" 18. ALLEN
" 19. ADAMS
" 20. WELLS
" 21. HUNTINGTON
" 22. WABASH
" 23. MIAMI
" 24. CASS
" 25. WHITE
" 26. BENTON
" 27. WARREN
" 28. TIPPECANOE
" 29. CARROLL
" 30. CLINTON
" 31. HOWARD
" 32. TIPTON
" 33. GRANT
" 34. BLACKFORD
" 35. JAY
" 36. RANDOLPH
" 37. DELAWARE
" 38. MADISON
" 39. HAMILTON
" 40. BOONE
" 41. MONTGOMERY
" 42. FOUNTAIN
" 43. VERMILION
" 44. PARKE
" 45. PUTNAM
" 46. HENDRICKS
" 47. MARION
" 48. HANCOCK
" 49. HENRY
" 50. WAYNE
" 51. UNION
" 52. FAYETTE
Niles
South Bend
Elkhart
Goshen
Kendallville
Auburn
Angola
Fort Wayne
Huntington
Wabash
Peru
Logansport
Delphi
Flora
Kokomo
Marion
Hartford
Portland
Dunkirk
Muncie
Anderson
Elwood
Tipton
Frankfort
Lafayette
Crawfordsville
Covington
Williamsport
Attica
Rockville
Terre Haute
Vincennes
Bluffton
Van Wert
Lima
Celina
Greenville
Piqua
Troy
Defiance
Napoleon
Ottawa
Wauseon
Waterloo
Plymouth
Warsaw
Winamac
Rochester
Knox
Valparaiso
Crown Point
Michigan City
Laporte
Mishawaka
Plymouth
Kouts
Hebron
Chalmers
Monticello
Rensselaer
Kingsbury
Medaryville
Francesville
Bremen
Ligonier
Kendallville
Wabash River
Maumee River
Tippecanoe River
Kankakee River
Wabash River
Mississinewa River
Mississinewa River
Mississinewa



MAP OF INDIANA SHOWING THE RELATION OF THE DRIFT TO THE ORDINARY WELLS

BY FRANK LEVERETT, 1897

WELLS OF NORTHERN INDIANA.[1]

By FRANK LEVERETT.

GENERAL STATEMENT.

The data included in this discussion of Indiana wells were collected ainly during the prosecution of the study of the glacial drift by the riter under the direction of Prof. T. C. Chamberlin. The portion the State north of the latitude of Terre Haute and Brookville, nbracing about two-thirds of the area, has been examined much ore thoroughly than the portion south of that line. In the former strict nearly every township has been traversed and close attention ven to the conditions for obtaining wells. In the latter district only few hasty trips have been made. Consequently the discussion for e northern two-thirds of the State is much fuller than for the southn third. In addition to material personally collected a considerable nount is taken from reports of the Indiana geological survey, many ell sections having been published in those reports. Such material accredited to the original sources.

A portion of the State, as has long been known, lies outside the acial boundary. The approximate position of the boundary has en determined in part by Prof. G. F. Wright, in connection with the nited States Geological Survey, and in part by members of the India geological survey. Professor Wright's location of the glacial undary in the southwestern part of the State appears to be less curate than in the southeastern part. In the former district it has en found by members of the Indiana survey and also by Prof. J. C. ranner to be in places at least 20 miles back from the limits of the ift, but in the latter district the writer's studies have sustained, in e main, the tracing made by Professor Wright, no drift having been ted more than 6 or 8 miles outside his line. The latest results conrning the position of the glacial boundary are embodied in the acial map of Indiana and Ohio here presented (Pl. I). It will be served that the boundary lies south of the Ohio River as far west the vicinity of Louisville, Kentucky. Just above this city it crosses e Ohio and turns abruptly northward, passing through western arke, northeastern Washington, central Jackson, and eastern Brown

The length of a single paper in this series being limited by law to 100 pages, it is necessary to ide this paper into two parts and issue them separately. The last part, relating to the south portion of the State, will be published as Water-Supply and Irrigation Paper No. 26 (in press).

PLATE NUMBERS TRANSPOSED.

counties, Indiana, a distance of about 50 miles. The course is then westward for about 30 miles, through northern Brown and northern Monroe counties. In northwestern Monroe County it makes an abrupt turn to the south and leads through western Monroe, eastern Greene, western Martin, and northwestern Dubois counties. From southwestern Dubois County the course is south of west across Pike, Gibson, and Posey counties to the extreme southwestern corner of the State.

The average thickness of the drift in the State is estimated to be about 130 feet. It consists of a complex series of sheets, differing widely in age and in structure. These sheets are made known largely by the well sections which appear in this report, for streams and natural exposures seldom reach the bottom of the drift. From a study of the drift sheets of Indiana and neighboring States, the glacial succession has been worked out sufficiently to determine the leading stages of glaciation and deglaciation.

It appears that the sheet which forms the surface of much of the glaciated portion of Illinois, and which is known as the Illinoian drift sheet, is exposed only in a few counties in Indiana, in the southwestern and southeastern portions of the State, the remainder being covered by later deposits of drift. The thickness where this single sheet is exposed to view averages scarcely more than 30 feet, or less than one-fourth the estimated average for the State.

In Illinois and Iowa there is a sheet of glacial drift exposed to view which was apparently deposited contemporaneously with the great silt deposit of the Mississippi Valley, to which the German term *loess* has been applied. This sheet has not yet been identified in Indiana, but the Illinoian drift sheet, where exposed to view, is capped by a deposit of loess-like silt, several feet in thickness, which was in all probability deposited at the time of the loess deposition of the Mississippi Valley and of the correlative drift sheet, which from its development in Iowa is known as the Iowan sheet. The Iowan drift sheet may be present underneath later drift sheets in the northern portion of Indiana.

The next important stage of glaciation has been termed the Wisconsin, chiefly because of its extent and early recognition as a later sheet of drift in the State of that name. It covers the northeastern fourth of Illinois and extends south into Indiana, about to the latitude of Terre Haute and Brookville. The border is somewhat irregular, but departs scarcely more than 20 miles to the north or south of a line leading eastward from Terre Haute to Brookville, the greatest departure being in the western part of the State, where it is deflected northward near the line of the Wabash River about 20 miles. The course of this boundary is indicated on the glacial map (Pl. I). It passes near the towns of Terre Haute, Rockville, Greencastle, Mooresville, Columbus, and Brookville. The loess referred to above covers

the drift to the south of this line, but passes beneath the drift of Wisconsin age to the north of it. In the discussion of counties near this boundary numerous sections are given showing that the Wisconsin drift is much fresher than that which underlies it. The difference in age is known not only by difference in induration of the bowlder clays, but also by the presence of a distinct soil formed on a land surface between the withdrawal of the ice of Illinoian age and the glacial advances which resulted in the deposition of later drift sheets. A soil is found between the Illinoian drift sheet and the capping of loess as well as between the latter and the overlying Wisconsin drift.

From the time the ice sheet occupied the outer border of the Wisconsin drift down to its final withdrawal from the State, it was apparently subjected to considerable oscillation of front and also to a shifting in direction of movement. In the earlier stages the movement was apparently nearly southward across northern Indiana, but in the later stages there was a change to a southwestward movement. There is a growing tendency among the glacialists who are familiar with this region to separate the two movements, giving the name Early Wisconsin to the southward and Late Wisconsin to the southwestward movement.

During the southwestward movement Indiana was covered by ice nearly as far south as during the southward movement, but Illinois was invaded to a comparatively short distance at that stage of glaciation. During this southwestward movement the ice sheet was divided into a series of lobes whose extent and characteristics have been set forth by Chamberlin in the Third Annual Report of this Survey. At the junction of these lobes great interlobate moraines were formed. One of these, occupying the line between the Saginaw and Erie-Maumee lobes, leads from the northeastern corner of Indiana southwestward to Cass County. These interlobate moraines are generally more prominent than those formed at the ends of the lobes. The one just referred to has a few knolls which rise nearly 200 feet above the neighboring basins, though the oscillations in level between knolls and basins are generally less than 50 feet. One of the most prominent moraines formed at the end of an ice lobe is the Valparaiso, situated north of the Kankakee Basin, in northwestern Indiana, and so called from the city of Valparaiso, located on it. It was formed by the Lake Michigan glacier at a time when that glacier extended but little beyond the limits of the shore of Lake Michigan. Another prominent moraine, known as the Maxinkuckee, leads southward from Michigan past South Bend to Lake Maxinkuckee, and thence eastward to join the great interlobate moraine formed between the Saginaw and Erie lobes. It was formed by the Saginaw ice lobe at the time it first became clearly differentiated from the Erie lobe. These and several other moraines are discussed in connection with the well sections which have been sunk in them. Further discussion at this point seems, therefore, unnecessary.

It will be observed that the structure of the drift is intimately related to the position with reference to the ice margin at the time of deposition. Much of the drift deposited under the margin of the ice is a compact, bowlder clay, to which the Scottish term *till* is applied, while that deposited at the end, or carried beyond the ice by waters issuing from it, consists largely of sand and gravel, or if the issuing waters were sluggish the deposits may be of a silty character.

In the discussion of the counties which border the Kankakee marsh attention is called to a deposit of sand which covers the glacial drift. It was formed apparently by a glacial lake which occupied that region for a brief period while the ice sheet was still blocking the Wabash Valley to the south. This lake was long since recognized by Bradley and named Lake Kankakee.[1] A lake formerly extended beyond the present shore of Lake Michigan into the northwestern counties of Indiana, to which the writer has recently applied the name Lake Chicago.[2] It had southwestward discharge, through what is known as the Chicago outlet, into the Des Plaines River and thence to the Illinois and Mississippi. This lake also has left deposits of sand on the counties of Indiana which it covered. In the northeastern part of Indiana there is also an old lake area with well-defined beach lines, occupied by the glacial Lake Maumee,[3] which had southwestward discharge, through the "Fort Wayne outlet," into the Wabash River. This lake bottom is covered less thickly by deposits of sand and silt than the bottoms of Lakes Kankakee and Chicago.

The discussion begins at the northwestern corner of the State, and counties are taken up in order in tiers, the first tier running west to east, the second from east to west, the third from west to east, etc., following nearly the numbering of the counties on the maps. In the case of the southern counties, the data are so meager that they are considered in groups under the head of the immediately subjacent rock formation.

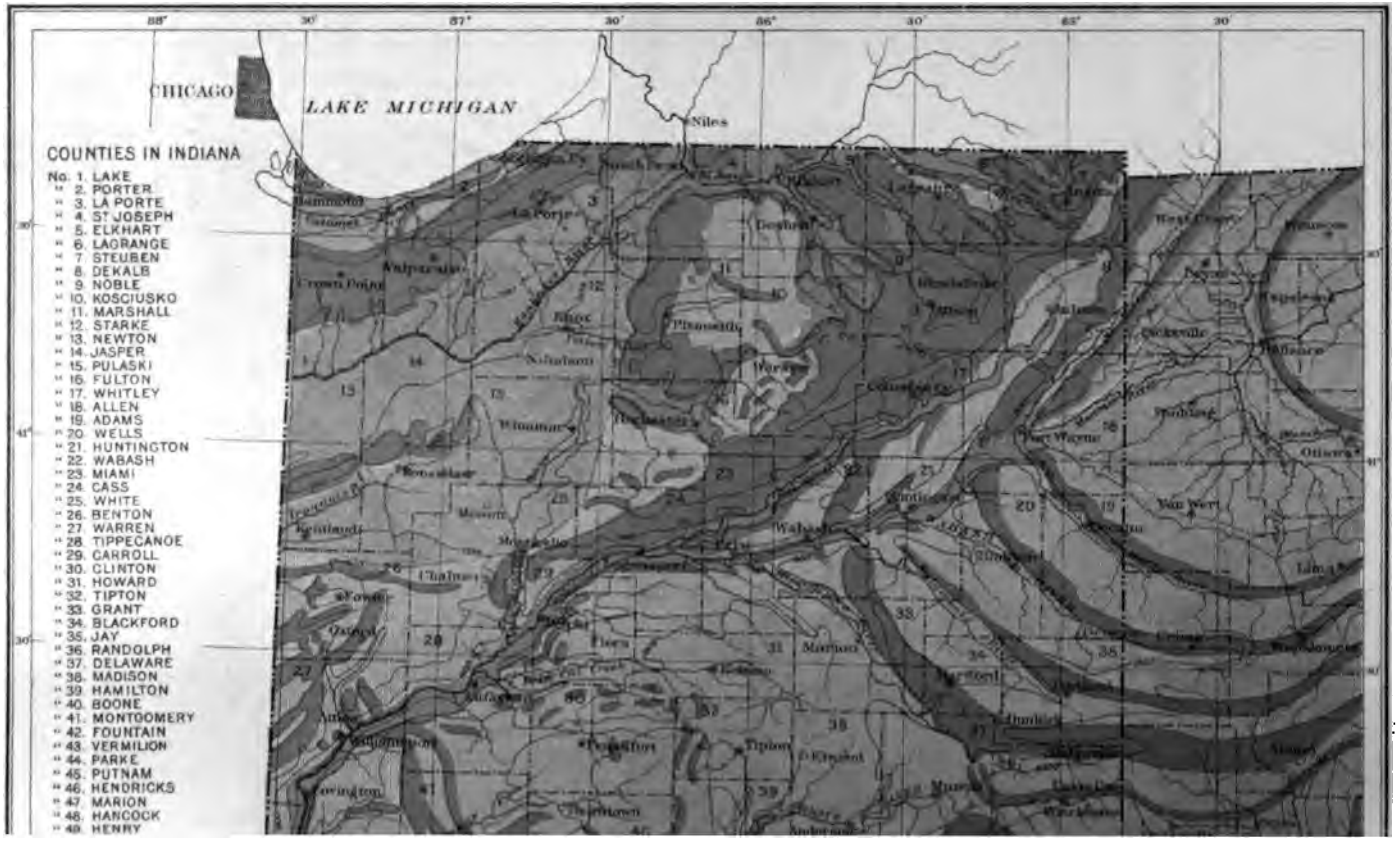
The records of wells here presented should not be understood to indicate the usual depth, for the majority of wells probably are less than 25 feet in depth. They represent the deepest concerning which information could be obtained, and are selected because such wells convey more information con erning the drift structure and its capacity for supplying wells than the shallow ones. The relation of the drift to the ordinary wells is set forth in Pl. II.

The borings for natural gas, which are especially numerous in the central and eastern parts of the State, often throw light upon the structure of the deeper portion of the drift and its water resources where deep water wells have not been made. They also throw considerable light upon the water horizons in the underlying rock strata. Such information as was obtained from the records of these borings is

[1] *Geol. of Illinois*, Vol. IV, 1870, pp. 226-229.

[2] *Geol. and Nat. Hist.* Survey, Chicago Acad. of Sciences, Bull. No. 2, 1897, pp. 64-66.

[3] *C. R. Dryer, Sixteenth* Ann. Rept. Indiana Geol. Survey, 1888, pp. 107-114.


CHICAGO
LAKE MICHIGAN
COUNTIES IN INDIANA
No. 1. LAKE
" 2. PORTER
" 3. LA PORTE
" 4. ST JOSEPH
" 5. ELKHART
" 6. LAGRANGE
" 7. STEUBEN
" 8. DEKALB
" 9. NOBLE
" 10. KOSCIUSKO
" 11. MARSHALL
" 12. STARKE
" 13. NEWTON
" 14. JASPER
" 15. PULASKI
" 16. FULTON
" 17. WHITLEY
" 18. ALLEN
" 19. ADAMS
" 20. WELLS
" 21. HUNTINGTON
" 22. WABASH
" 23. MIAMI
" 24. CASS
" 25. WHITE
" 26. BENTON
" 27. WARREN
" 28. TIPPECANOE
" 29. CARROLL
" 30. CLINTON
" 31. HOWARD
" 32. TIPTON
" 33. GRANT
" 34. BLACKFORD
" 35. JAY
" 36. RANDOLPH
" 37. DELAWARE
" 38. MADISON
" 39. HAMILTON
" 40. BOONE
" 41. MONTGOMERY
" 42. FOUNTAIN
" 43. VERMILION
" 44. PARKE
" 45. PUTNAM
" 46. HENDRICKS
" 47. MARION
" 48. HANCOCK
" 49. HENRY
Niles
La Porte
Crown Point
Kokomo
Wabash
Marion
Muncie
Anderson
Tipton
Frankfort
Lafayette
Terre Haute
Vincennes
Ottawa
Lake

MAP OF INDIANA SHOWING THE PLEISTOCENE DEPOSITS

BY FRANK LEVERETT, 1897

ıere presented, though it is not so full as could be desired. In this onnection it may be remarked that information concerning water supıly in the deeper part of the drift and in the underlying rocks is very ıeager, compared with the knowledge concerning the surface portion ıf the drift. Information concerning these deeper beds has been ;reatly extended since the discovery of natural gas. In many nstances borings which failed to obtain gas have been subsequently urned to use because of the water which they yield. Thus a resource vhich was overlooked in the excitement incident to the discovery of ;as is likely to receive proper development in the near future.

DETAILED DISCUSSION OF WELLS.

LAKE COUNTY.

General statement.—Lake County, situated in the extreme northvest corner of Indiana, has for its northern border the shore of Lake Michigan and for its southern border the Kankakee River. It extends ıastward from the State line about 16 miles, and has an area of 500 ıquare miles.

The Valparaiso moraine leads west to east across the central porion of the county, and has a width of about 10 miles. It is a somevhat elevated rolling tract, standing 100 to 200 feet higher than Lake Michigan. Wells are sunk to considerable depth, few wells being ıbtained at less than 25 feet, while many are 50 to 100 feet or more. These wells are obtained usually in beds or pockets of sand or gravel ıetween till sheets.

North of the moraine there is a low plain, much of which was covıred by Lake Chicago, the glacial predecessor of Lake Michigan, which ıad its outlet southwest to the Des Plaines River. For several miles ıouth from the present shore of Lake Michigan the sand deposits of his old lake bottom are so heavy that wells 10 to 40 feet in depth are ıften obtained without reaching the underlying till. But near the noraine the sand becomes thinner, and is in places but a mere skim ;oating. On this portion of the plain wells are carried to as great lepth as on the moraine, and water is obtained in gravel deposits in he till.

South of the Valparaiso moraine is a narrow strip of gravelly land 'ormed by the waters escaping from the ice at the time it occupied he moraine. On this plain the wells usually obtain their water at lepths of 10 to 25 feet, without reaching the bottom of the gravel leposit. Extending several miles north from the Kankakee, in the ıouthern part of the county, there is a marsh too wet for settlement.

The glacial deposits of Lake County are so heavy that wells may ıe obtained without entering the underlying rock. Borings for arteian water or for natural gas, made at Hammond, Hobart, and Crown 'oint, penetrate 110, 160, and 176 feet of drift, respectively. From hese borings it appears that the drift of this county may have an *verage thickness of about 150* feet.

Individual wells.—At Hammond many wells are driven to a depth of 10 to 18 feet in sand, entering water at about 8 feet. Filters are used in some cases. Below the sand is a nearly solid bed of blue till, 90 feet in thickness, which contains but little water. At about 110 feet Lockport (often called Niagara) limestone is entered, which furnishes water containing sulphureted hydrogen. The artesian well boring at the distillery, 1,900 feet in depth, is discussed elsewhere.[1]

At Whiting a portion of the water supply is furnished by shallow wells in the beach sand, 20 feet or less in depth. A portion of the water supply is pumped from Lake Michigan by the Standard Oil Company and made use of by many families.

The villages of Gibson, Hessville, Tolleston, Liverpool, and Lake all depend upon shallow wells driven or dug to a depth of 10 to 40 feet in the surface sand. In some cases contamination is liable to occur from privy vaults and other sources.

At Hobart the wells 15 to 25 feet in depth usually penetrate a few feet of clay, which probably prevents contamination of water. A well at Owens's brickyard reaches the underlying rock and has the following section:

	Feet.
Yellow silt, pebbleless, calcareous	10
Blue silt, pebbleless, calcareous	50
Blue till, hard and stony	97
Hard rock	3
Total	160

Mr. Leavenworth's well, 3 miles west of Hobart, depth 80 feet, is above the limits of the lake deposit and is mainly through blue till. Several wells in that vicinity 50 to 100 feet deep do not reach the bottom of the drift.

Wells in the vicinity of Ainsworth, on the north slope of the Valparaiso moraine, often exceed 50 feet. After penetrating yellow till 10 or 12 feet they are mainly through blue till.

In the vicinity of Crown Point, near the crest of the Valparaiso moraine, there are several tubular wells 75 or 80 feet in depth, mainly through blue till.

Tubular wells in the vicinity of St. John, near the crest of the moraine, obtain water at 50 to 100 feet in sand beds included in the till.

Wells on the south slope of the Valparaiso moraine usually enter a water-bearing sand or gravel after penetrating a few feet of till, water being more easily obtained there than on the crest or north slope.

A well at the powder factory, 1 mile west of Miller Station, in the northern part of the county, 75 feet in depth, entered a bed of molluscan shells at the bottom of the sand, 40 feet below the surface and probably several feet above the level of Lake Michigan. Below the shells a bluish silt or fine sand is penetrated 35 feet.

[1] *See Eighteenth* Ann. Rept. U. S. Geol. Survey, 1896-97, Part IV, 1897, p. 489.

PORTER COUNTY.

General statement.—Porter County is situated just east of Lake ounty, and has Lake Michigan for its northern and the Kankakee :iver for its southern border. It is 15 miles in breadth, 25 to 30 iiles in length, and has an area of 410 square miles.

The Valparaiso moraine traverses the county from southwest to ortheast, slightly north of the center, and has a width of about 8 iiles. It reaches a higher elevation in this county than in Lake, ome points being fully 250 feet above Lake Michigan. It includes everal lakes, one of which furnishes the water supply for Valparaiso, he county seat. In the western portion of the county till predomiates as far as wells penetrate—150 to 200 feet. But in the eastern ortion of the county, where the moraine reaches its highest elevaion, it contains a large amount of dry sand and gravel, and strong vells are obtained only by penetrating to a depth of 100 to 160 reet.

Extending north from the moraine there is a low plain reaching to he Calumet River, which is underlain, as a rule, by till, but is coated uite generally with sand. In this plain wells vary greatly in depth, ut are usually 20 to 40 feet.

North of the Calumet River there is a narrow ridge of till forming feeble morainic belt. Its width seldom exceeds 1 mile and its elief is but 30 to 50 feet. Along this ridge wells are frequently 75 o 125 feet in depth, it being one of the most difficult places in the ounty to obtain water.

North of this till ridge there are heavy deposits of lake sand, renderng a portion of the district a barren waste. The sand is drifted by he wind into dunes 100 to 200 feet in height. This belt, however, ccupies only a breadth of 2 or 3 miles, though extending the entire vidth of the county. This district is largely uninhabited. In the arms along its border next the till ridge wells usually obtain water efore reaching the bottom of the sand at depths of 25 to 50 feet.

South of the Valparaiso moraine the formations are like those of ake County, there being a gravel plain several miles in width, xtending on the south into the uninhabited Kankakee marsh. The noraine extends spurs out into this gravel plain nearly to the borders f the marsh.

The glacial deposits of this county, like those of Lake, are so heavy hat wells obtain water without entering the underlying rock. Borngs for natural gas made at Valparaiso, Hebron, and Boone Grove enetrate 126, 108, and 140 feet of drift, respectively, while a boring or an artesian well in the northeast part of the county has 245 feet f drift. It appears from these borings that the average thickness of he drift in this county may slightly exceed 150 feet. Several wells long the Valparaiso moraine, as indicated below, reach a depth of ver 150 feet *without entering rock.*

Individual wells.—The Blair artesian well, in the extreme northeast corner of the county (sec. 1, T. 37, R. 5 W.), has the following section:

	Feet.
1. Sand with peaty beds near bottom	30
2. Adhesive blue clay with thin sand beds	55
3. Gravel and sand with water	9
4. Blue clay like No. 2	110
5. Gravel with water	6
6. Blue clay like No. 2	35
7. Shale (?)	1
8. Hard gray limestone	180
9. Soft rock, subject to caving in places	66
10. Hard limestone	372
Total	864

The well mouth is about 20 feet above Lake Michigan, or 600 feet above tide. Flows occur at 68, 72, 78, 80, 85, 95, and about 200 feet in the drift. The flows were weak until a depth of about 85 feet was reached, when a strong flow with head 18 feet above the surface was obtained. The flow from 200 feet is also strong. The water from all the veins in the drift is fresh and of good quality. At a depth of 370 feet sulphur water was struck, which gradually increased in strength in the next 150 feet. The head is, however, 10 feet lower than that from the drift deposits. The combined flow is about 400 barrels per hour and the temperature 62° F.

At Furnessville, on a beach line in the north part of the county, a well 80 feet in depth penetrated beach sand 25 feet, at the base of which logs were encountered. Below the logs is a blue clay, probably till, which was penetrated about 50 feet when a water-bearing sand and gravel was struck.

A well on the beach line immediately north of Calumet River, near the east line of the county, obtains water at the base of the beach sand at 23 feet. Neighboring wells south of the beach line enter a blue clay, nearly pebbleless, at 3 to 5 feet, and in some cases penetrate this to a depth of 50 feet before obtaining water.

At Chesterton and Hageman, in the northern part of the county, wells are seldom more than 20 feet in depth, and usually obtain water at 10 or 12 feet. As they are mainly through sand, there is danger of contamination.

Records of several wells were obtained along the till ridge north of Calumet River, near Chesterton. They penetrate yellow till 18 or 20 feet, below which is a blue till, hard and stony, which is usually penetrated 40 or 50 feet before a water-bearing sand or gravel can be obtained. In one well a dry sand, 15 or 20 feet in thickness, was found between the yellow and blue tills, and the blue till there afforded no water until it had been penetrated 70 feet. The depth of the well is 125 feet.

From the Calumet River south to the Valparaiso moraine water is ually obtained at 8 to 25 feet. In some cases surface clay a few et in depth is penetrated, but quite as often the wells are almost tirely in sand or gravel. In the western part of the county, how-er, in the vicinity of Wheeler and northward for 4 or 5 miles, this ain is underlain by a compact till, and wells are in some places fficult to obtain. There is a small till ridge leading north from 'heeler, on which it is especially difficult to obtain wells. Several rings have been made along the ridge to a depth of 100 to 200 feet. one case a boring was abandoned at 200 feet without obtaining ater. In another case water was obtained after penetrating 155 feet till. Another well found water after penetrating 115 feet of till. plain known as Twenty Mile Prairie, lying north and east of the ll ridge just mentioned, is underlain by a very fine sand, so that ells have been carried to a depth of 150 feet before entering coarse nd or gravel. On this prairie there is usually:

	Feet.
Yellow till	7 to 10
Blue till	20
Fine gray quicksand	75 to 125

Passing to the Valparaiso moraine, we find in the western part of e county a region in which strong wells are difficult to obtain, cept in a few favored localities, there being a very compact till, milar to that on the plain north of the moraine. At Smalls Cross-g, 3 miles southwest of Wheeler, a well penetrates the following ds:

	Feet.
Yellow till	8 to 10
Blue till	35 to 40
A fine sandy deposit, in places resembling till	150
Coarse water-bearing sand	19
Total	219

Mr. Currier's well, 1 mile south of Wheeler, in which no good vein water was found, penetrates—

	Feet.
Yellow till	15
Blue till, soft	140
Blue till, hard	39
Total	194

On the south slope of the moraine in the western part of the county, ter-bearing sand or gravel is usually found at about 40 or 50 feet. Following the moraine northeastward to Valparaiso, the till is found give place in its lower portion to beds of sand and gravel. In the thern part of Valparaiso, and also for some distance south of the y, wells often obtain water at a depth of 25 feet. Occasional deeper lls in that district show a large amount of coarse sand or fine gravel low a few feet of surface till. On the elevated ground in the north-

ern part of Valparaiso wells usually penetrate about 30 or 40 feet of yellow and blue till, underlain by dry sand, which is often penetrated 50 or 60 feet before water-bearing sand or gravel is reached. The drift is of this character for several miles northeast from Valparaiso, and wells are often carried to a depth of 130 to 160 feet before obtaining water. For example, a boring at Jackson Center penetrates—

	Feet.
1. Yellow till	20
2. Dry sand	90
3. Blue-gray clay, tough and spongy	45 or 50
4. Water-bearing sand	10
Total	170

In some wells near Jackson Center a water-bearing sand is found at the top of the blue clay (No. 3) at about 120 feet. Where this is too fine to be easily screened the wells continue through the blue clay, obtaining water in coarse sand at 160 to 175 feet. Near Flint Lake the wells are 100 to 130 feet deep and obtain water in the sand above the blue clay.

In southern Porter County a boring for gas at Boone Grove penetrates—

	Feet.
1. Yellow loamy clay	8
2. Water-bearing sand	92
3. Hard blue till	40
4. Black shale (Devonian)	125
5. Limestone	15
Total	280

In the gravel plain south of the Valparaiso moraine wells usually obtain water at 10 to 20 feet, without penetrating any till. The gravel in places, however, does not coat the sheet of till that lies outside the moraine. Thus at Kouts Station there is a surface till 18 or 20 feet, beneath which water is obtained in a deposit of gravel at 25 to 35 feet. At Hebron, also, wells encounter till, and water is found at about 40 feet.

The Kankakee marsh extends several miles north of the river in the southern part of the county, and this area is so generally covered with water that it is unfit for settlement.

LAPORTE COUNTY.

General statement.—Laporte County is situated immediately east of Porter. It fronts for a few miles on Lake Michigan, on its northern boundary, but that boundary is formed mainly by the State line. The southern boundary is formed mainly by the Kankakee River, but in the southeastern part the county extends a few miles beyond the river. The width of the county is 21 to 23 miles and the length is 22 to 32 miles, being greatest at the western border. The area is 540 square miles.

The Valparaiso moraine leads northeastward across the northwestern portion of the county and has a width of about 6 miles. Its crest has an elevation of 225 to 300 feet or more above Lake Michigan. It encloses several lakes, from one of which the city water supply of Laporte is obtained. It rises very abruptly on its northwest border above the low plain that lies between it and Lake Michigan, but on its southeast border a gravel outwash from the moraine is built up nearly to the level of the crest, and the descent is gradual from the moraine to the Kankakee marsh. The marsh stands fully 100 feet above Lake Michigan in eastern Laporte County and about 75 feet at the western border of the county. It is, therefore, 150 to 200 feet or more below the crest of the moraine. The gravel plain makes a descent of 75 or 100 feet in the interval of 8 or 10 miles between the moraine and the marsh.

On the low plain bordering Lake Michigan, in the northwestern part of the county, there are a series of narrow till ridges or feeble moraines which govern the drainage of that region to a marked degree, though having a relief of but 30 to 50 feet. On the immediate border of the lake there are prominent dunes, rising in places to a height of 150 feet above lake level.

The glacial deposits are so heavy that ordinary wells are obtained without entering the underlying rock. A boring for gas at Lacrosse, in the southern part of the county, is reported to enter rock at 38 feet. The gas boring at Laporte, the county seat, penetrates 295 feet of drift. The gas boring at Michigan City penetrates 250 feet, while the artesian well at the penitentiary at Michigan City penetrates only 170 feet. These borings indicate that the thickness varies greatly, and are scarcely sufficient to justify an estimate of the average thickness for the county.

Individual wells.—The gas-well boring at Michigan City has the following section:

	Feet.
Dry surface sand	15
Quicksand	10
Gravel	5
Blue till, pebbly and rather hard	167
Gravel containing cobble and bowlders	15
Sand and gravel, with occasional large stones	40
Limestone, with thin beds of shale, extending to the bottom of the well	570
Total	819

At a depth of 210 feet a flow of fresh water was obtained from the drift, with a head 15 feet above the surface. The strength of the flow increased downward to the rock at 250 feet. A strong flow of sulphur water was obtained from the limestone at about 450 feet.

here presented, though it is not so full as could be desired. In this connection it may be remarked that information concerning water supply in the deeper part of the drift and in the underlying rocks is very meager, compared with the knowledge concerning the surface portion of the drift. Information concerning these deeper beds has been greatly extended since the discovery of natural gas. In many instances borings which failed to obtain gas have been subsequently turned to use because of the water which they yield. Thus a resource which was overlooked in the excitement incident to the discovery of gas is likely to receive proper development in the near future.

DETAILED DISCUSSION OF WELLS.

LAKE COUNTY.

General statement.—Lake County, situated in the extreme northwest corner of Indiana, has for its northern border the shore of Lake Michigan and for its southern border the Kankakee River. It extends eastward from the State line about 16 miles, and has an area of 500 square miles.

The Valparaiso moraine leads west to east across the central portion of the county, and has a width of about 10 miles. It is a somewhat elevated rolling tract, standing 100 to 200 feet higher than Lake Michigan. Wells are sunk to considerable depth, few wells being obtained at less than 25 feet, while many are 50 to 100 feet or more. These wells are obtained usually in beds or pockets of sand or gravel between till sheets.

North of the moraine there is a low plain, much of which was covered by Lake Chicago, the glacial predecessor of Lake Michigan, which had its outlet southwest to the Des Plaines River. For several miles south from the present shore of Lake Michigan the sand deposits of this old lake bottom are so heavy that wells 10 to 40 feet in depth are often obtained without reaching the underlying till. But near the moraine the sand becomes thinner, and is in places but a mere skim coating. On this portion of the plain wells are carried to as great depth as on the moraine, and water is obtained in gravel deposits in the till.

South of the Valparaiso moraine is a narrow strip of gravelly land formed by the waters escaping from the ice at the time it occupied the moraine. On this plain the wells usually obtain their water at depths of 10 to 25 feet, without reaching the bottom of the gravel deposit. Extending several miles north from the Kankakee, in the southern part of the county, there is a marsh too wet for settlement.

The glacial deposits of Lake County are so heavy that wells may be obtained without entering the underlying rock. Borings for artesian water or for natural gas, made at Hammond, Hobart, and Crown Point, penetrate 110, 160, and 176 feet of drift, respectively. From these borings it appears that the drift of this county may have an average thickness of about 150 feet.

Individual wells.—The gas-well boring at South Bend is reported by Dr. Phinney[1] as follows:

	Feet.
1. Sand	25
2. Gravel	20
3. Clay	30
4. Sand	25
5. Gravel	37
6. Shales	238
7. Limestone	805
8. Limestones and shales	220
9. Utica shale	185
10. Trenton limestone	85
Total	1,670

Salt water was struck at 375 and 610 feet, and also in the Trenton at 1,670 feet. From the latter horizon it rose within 200 feet of the surface.

The city of South Bend obtains its water supply from about 30 flowing wells, situated along the valley of the St. Joseph River. The wells have a depth of 100 to 125 feet, with an average of about 110 feet. They are obtained in gravel beneath a bed of clay 20 to 40 feet in thickness, which acts as a cover and prevents contamination of the water. This clay is probably a water deposit rather than a glacial deposit, as it is described as entirely free from pebbles of any kind. The head is sufficient to carry the water above the surface only on the low ground along the river bottom, about 670 feet above tide. The gravel plain on which the city was built stands 20 to 30 feet or more above the height to which the water will rise.

The private wells in South Bend are often obtained without penetrating a bed of clay, and the ground water is usually at sufficient height to easily receive the leaching from cesspools. Such wells may become a source of danger to the health of the city.

Wells in the city of Mishawaka are also situated on a gravel plain, and are usually about 20 feet in depth, and the water stands within 12 feet of the surface. There appear in the majority of cases to be no beds of clay or other impervious material to prevent contamination of the water from cesspools. The St. Joseph River, drawn upon for fire protection and irrigation, might easily be made to furnish the supply for domestic use.

Wells on Portage Prairie, a gravel plain in the northwest part of the county, are usually 60 or 80 feet, and seldom less than 40 feet, in depth. A well on this prairie, at Mr. Womer's, near the State line, 174 feet in depth, has the following section:

	Feet.
1. Sand and gravel, with water near bottom	67
2. Blue clay and blue quicksand	99
3. Gravel and sand, with water	8
Total	174

[1] *Eleventh Ann. Rept. U. S. Geol. Survey*, 1889-90, Part I, 1891, p. 734.

Wells on the gravel plain west of the Maxinkuckee moraine, near North Liberty, usually enter clay below the gravel at about 20 feet. Two wells in the Potato Creek valley, east of North Liberty, obtain flows of water from depths of 45 and 70 feet.

Wells at Walkerton, also on the gravel plain, obtain water at 20 or 30 feet. The water surface is usually nearly 20 feet from the mouth of the well, but even this depth is not sufficient to insure freedom from contamination, there being no impervious stratum above the water vein.

A well on the gravel plain between Walkerton and North Liberty, in sec. 5, T. 35, R. 1 E., 160 feet in depth, is mainly through sand and gravel. The water rises nearly to the surface. Wells in that vicinity usually obtain water at 10 to 20 feet. One well near Walkerton, however, failing to obtain water in the surface gravel, passed through a pebbleless blue clay 50 feet in thickness, obtaining water at a depth of 70 or 80 feet.

Records of two deep wells were obtained on the Maxinkuckee moraine, about 8 miles south of South Bend, in sec. 17, T. 36, R. 2 E. One at Mr. Sweeney's, 131 feet in depth, after penetrating 22 feet of yellow till, passes through alternations of sand and blue till for about 100 feet before reaching the coarse gravel. The other well, at Mr. McClelland's, has a similar section and is 146 feet in depth.

On the plain east from the Maxinkuckee moraine many wells are but 15 to 20 feet in depth, and enter sand or gravel below till near the bottom. A few tubular wells are 80 or 100 feet in depth.

ELKHART COUNTY.

General statement.—Elkhart County is situated immediately east of St. Joseph County, on the north boundary of the State, and has an area of 470 square miles. Like St. Joseph County, a large portion of its surface is a gravel plain. The most extensive gravel plain is that along the St. Joseph River, in the northwest part of the county, which comprises an area of more than 100 square miles. There is also a gravel plain along the Elkhart River from the southeast corner of the county northwestward to the St. Joseph gravel plain. From the Elkhart gravel plain there are extensions southward into Kosciusko County, past New Paris and Milford, and southeastward into Noble County near Cromwell. This system of gravel plains comprises an area of fully 100 square miles. There is also a narrow gravel plain along Little Elkhart River, in the northeast part of the county. About half the county is, therefore, embraced in the gravel plains. The uplands consist of a till plain having an area of about 125 square miles, situated in the southwest part of the county, and of morainic belts, with small areas of nearly plain surface, occupying an area of about 150 square miles in the eastern and southern parts of the county. *The altitude of the gravel* plains is mainly below the 800-foot

contour. The uplands are mainly between 800 and 900 feet above tide, though small areas on the moraines in the eastern and northeastern parts of the county rise above 900 feet.

The thickness of the drift is known at three points—Elkhart and Goshen, and near New Paris, where it is 122, 162, and 90 feet, respectively. Should the rock surface of the entire county be as low as at these wells, the thickness of the drift would average fully 200 feet.

Individual wells.—At Elkhart the majority of wells are about 30 feet in depth. They penetrate sand and gravel for 20 feet or more, when a thin bed of clay is usually found as a cover for the water-bearing gravel. The mayor reports that he considers the clay a protection against contamination.

A few wells in Elkhart have been put down to a depth of 100 to 120 feet, which have sufficient head to overflow on the low ground near the streams. They penetrate usually a large amount of blue clay, and obtain water in a coarse gravel or cobble near the base of the drift. The water from these flowing wells is chalybeate.

At Goshen the city water supply is from a large well, 34 feet in depth and 30 feet in diameter. After penetrating a few feet of surface sand, it passes through 19 feet of blue clay, which is thought to insure its freedom from contamination. The water is obtained from a coarse sand beneath the blue clay. Its head is sufficient to bring it to the level of the well mouth.

One of the gas-well borings at Goshen was carried only to the Lockport limestone, where salt water was struck which rose to the surface. Another gas-well boring in that city was carried to a depth of 239 feet into the Trenton (Phinney). So far as known no use is made of the water from either of these wells.

In the vicinity of Middlebury, in the eastern part of the county, on the elevated portions of the moraine, tubular wells are usually 80 or 100 feet deep and penetrate a large amount of till.

A well on the border of the St. Joseph gravel plain in the northeast part of the county, in sec. 27, T. 38, R. 7 E., is mainly through gravel to a depth of 65 feet. Wells on this plain are usually but 15 or 20 feet.

Near New Paris, on the border of a morainic ridge, one well penetrates till to a depth of 110 feet, another, 1 mile north, 102 feet in depth, is mainly in sand and gravel. It is thought to have struck rock at about 90 feet. The morainic tracts in that vicinity usually contain more gravel and sand than till. Knolls are often composed largely of gravel. In the eastern and northeastern part of the county the morainic tracts contain a larger proportion of till.

The upland plain in the southwestern portion of the county is a continuation of the plain of southeastern St. Joseph County, and its wells have similar depth and structure, being through till for a distance of 20 or 25 feet, when a water-bearing sand or gravel is struck.

In some cases wells are obtained at about 10 feet in a bed of sand or gravel between the yellow and blue till. A few tubular wells have been sunk to a depth of 80 or 100 feet to a water-bearing sand or gravel below a sheet of blue till.

LAGRANGE COUNTY.

General statement.—Lagrange County is situated immediately east of Elkhart, on the north boundary of the State, and has an area of 384 square miles. It is one of the most elevated counties in the State, the greater part of its surface being above the 900-foot contour, while several tracts in the eastern part of the county, aggregating an area of perhaps 75 square miles, stand above the 1,000-foot contour. It has several small morainic belts, formed by the Saginaw lobe during its recession. It has also conspicuous morainic hills in its northwestern corner and on the eastern border of the county. There is one extensive gravel plain in the county, traversed by Pigeon River. It lies on the southwest border of a prominent Saginaw moraine, which occupies northeastern Lagrange County and passes northwestward into Michigan. This plain has a breadth of 2 to 6 miles and traverses nearly the entire width of the county, a distance of over 20 miles. Aside from this main gravel plain there are small areas underlain by gravel, one of which, known as the Haw patch, covers several square miles on the southern boundary of the county; another borders the Little Elkhart River in the western part of the county.

So far as known to the writer, the bottom of the drift has been reached by only one boring in this county, though it has been penetrated to a depth of 200 feet in several wells, and there are numerous wells 100 feet or more in depth. It is thought by Dr. C. R. Dryer[1] that a farm well in southwestern Lagrange County may enter black shale at 150 feet.

Individual wells.—On the strong morainic belt in the northwest part of the county the sharpest knolls are usually made up of gravel and sand, but those of gentler contour often contain till. The wells vary greatly in depth, some being but 15 or 20 feet, while others are 50 to 75 or more. On the overwash gravel plain south of this moraine wells are obtained in gravel at about 20 feet.

In a feeble moraine on the north side of Little Elkhart River, in the western part of the county, the wells are usually about 30 feet in depth; after penetrating 10 to 20 feet of till they enter sand. South of Little Elkhart River the wells are mainly through till. The average depth is 30 or 40 feet, but occasional wells reach 60 feet.

In the gravel plain along Little Elkhart River the wells are usually but 12 to 20 feet in depth, entirely through gravel and sand.

At Lagrange wells are obtained at about 100 feet at the city waterworks, but private wells seldom exceed 40 feet. The detailed record

[1] *See Eighteenth Ann.* Rept. Indiana Geol. Survey, p. 81.

of four of the waterworks wells, published by Dr. Dryer,[1] show alternations of sand and gravel with till, in which the thickest beds are but 20 to 25 feet. In well No. 1, 92 feet in depth, the gravel beds amount to 32 feet, the thickest bed being 12 feet in the upper part of the section. In well No. 2, 102 feet in depth, the gravel and sand amount to 47 feet, of which 17 feet occur at the bottom of the well. In well No. 3, 98 feet in depth, the sand and gravel amount to 34 feet. In well No. 4, 111 feet in depth, the sand and gravel amount to 33 feet. These wells are within the area of an ordinary town lot, and yet Dr. Dryer reports that the majority of strata can not be correlated in any two of them. In well No. 1 there are 6 beds of gravel; in wells Nos. 2–4, but 5 beds of sand and gravel.

A well near the jail at Lagrange penetrates—

	Feet.
1. Yellow till	8
2. Dry sand	30
3. Gravel with water	2
Total	40

A prospect boring for artesian water at the jail penetrated—

	Feet.
1. Yellow till	15
2. Blue till, with thin beds of sand	50
3. A brown mold or soil	4
4. Cemented gravel	5
5. Loose gravel with water	8
6. Blue till	45
7. Cemented gravel	8
8. Dry sand and gravel	70
Total	205

The boring was abandoned without obtaining a flow or reaching rock strata.

A well on a sharp morainic ridge about 4 miles east of Lagrange, 202 feet in depth, failed to find water. Another well on the same farm obtained water at 175 feet. Both wells were mainly through till. A neighboring well, only 62 feet in depth, obtained water in gravel below till at about 60 feet.

A well for Dr. Drake, one-half mile north of Valentine, has the following section:

	Feet.
1. Sandy yellow till	10
2. Blue till	41
3. Gravel with inflammable gas	3
4. Blue clay	46
5. Black muck with leaves and inflammable gas	6
6. Soft blue clay	24
7. Water-bearing sand, coarse at bottom	10
Total	140

[1] Eighteenth Ann. Rept. Indiana Geol. Survey, pp. 80-81.

A well in the southeast part of the county, one-half mile east of uth Milford, penetrates—

	Feet.
Sandy yellow till	20
Blue till	49
Very fine sand	20
Total	89

A similar fine sand was found in the well at Joseph Ryer's, 4 miles rth of Wolcottville, at 30 to 108 feet from the surface.

A well in the west part of Milford Township, 3 miles north of county ıe, 97 feet in depth, is largely through sand, gravel, and cobble.

In the vicinity of Pretty Lake and Long Lake, in Milford Township, the southeast part of the county, there are gravel plains on which ɔlls obtained water at 30 or 40 feet without reaching the bottom of e gravel. On bordering districts wells usually penetrate 25 to 40 et of till before reaching a water-bearing gravel.

Dr. Dryer reports the occurrence of coal in the drift beds in the uthwest part of the county at depths of 76 feet in two neighboring ɔlls under which beds of water-bearing gravel occur. A boring on neighboring farm is thought to strike shale at 150 feet.

STEUBEN COUNTY.

General statement.—Steuben County is situated in the extreme ɪrtheast corner of the State, having Michigan on its northern and ıio on its eastern boundary. On its western border is Lagrange ıd on its southern border Dekalb County. Its area is about 330 uare miles. The entire county, except a valley in its southeast rner, stands above the 900-foot contour, and a portion of it rises ɔve 1,100 feet, with occasional points 1,150 feet or more. It is timated that at least one-half of the county stands above the 1,000-ot contour. The great elevation of this county is not due to an ɔvated rock surface, but to the heavy accumulations of drift. Its ɔstern half is occupied by the great interlobate moraine formed tween the Erie lobe and the Saginaw lobe.

The topography of the western half of the county is of the knob-ıd-basin type, and several lakes are inclosed among the morainic ıolls and ridges. These lakes have the advantage of beautiful enery, pure water, and an excellent stock of fish. They are justly e pride of the inhabitants of the county. The largest lake, James ıke, has a length of 5 miles and an average width of half a mile. is situated near the north line of the county and surrounded by arp morainic knolls, which rise in places to a height of more than 0 feet above its surface. About this lake is a group of smaller ıes, among which Jimerson, Crooked, Loon, Center, and Gage lakes e worthy of mention. These lakes are all tributary to Fawn River ıd discharge northwestward. South of them is a chain of lakes,

which are traversed by Pigeon River. Pleasant Lake, the most conspicuous of these lakes, is about 1 mile in length and one-fourth mile in average width. Below this lake the stream passes through Golden Lake and Hogback Lake, the latter receiving its name from a precipitous ridge of bowlders and gravel which stands on its shore. In the extreme southwestern part of the county is another chain of lakes drained by Turkey Creek, a south tributary of Pigeon River. The largest of these, Big Turkey, has a length of about 2½ miles and a maximum width of one-half mile.

A moraine of the Lake Erie lobe traverses the eastern part of the county in a north to south direction. This has gentler contours than the interlobate moraine, but incloses several lakelets among its knolls and ridges. The most conspicuous are Clear Lake, in the northeast corner of the county, having an area of about 2 square miles, and Fish Lake, in the southeast corner, having an area of a little more than 1 square mile. These lakes are tributary to the St. Joseph-of-the-Maumee River. A detailed discussion of the several lakes just mentioned, with observations on the size, depth, and character of their bottoms, is presented by Dr. Dryer in his report on Steuben County.[1] To this report the writer is indebted for the data concerning the dimensions of the lakes. It is suggested by Dr. Dryer that the city of Fort Wayne could find no better source of water supply than these lakes in the southern part of Steuben County.

The only level land in the county of notable extent is found in a narrow plain lying between the interlobate moraine and the Erie moraine just mentioned. It has a breadth of 2 to 5 miles and extends the entire length of the county.

The valleys of Fawn River, Pigeon River, Turkey Creek, and Fish Creek traverse gravelly districts, in which wells may be obtained at comparatively slight depth. The interlobate moraine varies greatly in the constitution of its drift and in the depth at which water may be obtained. Wells are not, however, as a rule, difficult to obtain, their average depth being not more than 30 or 40 feet. On the Erie moraine, in the eastern part of the county, and also on the plain west of it, the drift is largely a compact till, and the wells are not infrequently put down to a depth of 100 feet or more.

So far as known no wells in the county have reached rock, but the greatest ascertained depths are not more than 160 to 175 feet.

Individual wells.—Wells in the vicinity of Angola present much variety in material penetrated. Immediately north of the city is a small district where they obtain water at depths of 20 or 30 feet. Some wells in the city also obtain water at slight depth after penetrating a few feet of till. Several wells are put down to depths of 80 to 100 feet

[1] Seventeenth Ann. Rept. Indiana Geol. Survey, 1891, pp. 120-134.

or more. A well at the Tri-State Normal College has the following section:

	Feet.
1. Yellow till	20
2. Blue till	8
3. Reddish gravelly clay, with thin beds of sand	75
4. Gravel with water	6
Total	104

A well on the moraine west of Angola, near Fox Lake, 130 feet deep, was mainly in till. A well 3 miles north of Angola, of the same depth, was mainly in sand and gravel. On a gravel plain on south side of Turkey Lake two wells have a depth of 114 feet, largely through till after penetrating a few feet of surface gravel. A well on an elevated part of the moraine west of Turkey Lake in section 33, Salem Township, 90 feet in depth, was mainly in till. A well 2 miles southeast of Salem Center, 160 feet in depth, penetrates yellow or gray till 40 feet, below which it was mainly in blue till.

Near Hudson wells 80 feet in depth penetrate about 30 feet of sand and gravel and then enter blue till. A well 3 miles west of Hudson, 100 feet in depth, was mainly till.

Wells near Metz, in the eastern part of the county, are frequently 100 feet in depth, mainly till. One well 2 miles southwest of Metz, 103 feet in depth, is in till to within 2 to 3 feet of bottom.

Wells in the vicinity of Fish Lake, in the southeastern part of the county, obtain water in gravel at 15 to 25 feet. But within 2 miles back from the lake are frequently through till to a depth of 100 feet and occasionally 150 feet.

DEKALB COUNTY.

General statement.—Dekalb County is situated on the eastern boundary of the State, immediately south of Steuben. It has an area of 370 square miles. Its northwestern corner is occupied by the interlobate moraine of the Erie and Saginaw lobes. The higher part of the moraine rises above the 1,000-foot contour, but its eastern border stands only about 900 feet. The greater part of the county is occupied by a till plain, standing about 850 to 900 feet above tide. In the southeastern part, on the borders of the St. Joseph-of-the-Maumee, are two moraines of the Erie lobe, but they rise scarcely 25 feet above the border plains.

Over the greater part of the county there is a sheet of till extending to an average depth of about 50 feet from the surface, in which water is seldom obtained in large amount. Below this sheet of till the drift appears to be very largely sand and gravel and yields an unlimited amount of water. There are narrow belts or small areas, usually bordering the valleys, in which water may be obtained in gravel or sand at comparatively slight depth.

The thickness of the drift in this county has been determined at four villages, as follows: Butler, 378 feet; Waterloo, 365 feet; Auburn, 280 feet; Garrett, 318 feet. A boring in the extreme southeastern part of the county, near Spencerville, was carried to a depth of 198 feet without reaching rock. A boring at Kendallville, just west of this county, penetrated 485 feet of drift. These borings, together with the four which strike rock within the county, have sufficiently wide distribution to justify the interpretation that the drift of the county probably averages over 300 feet in thickness, and may possibly average 400 feet. The four borings which enter the rock are on a till plain which stands more than 100 feet below the elevated northwestern portion of the county. The rock floor in but one instance (at Auburn) stands above the level of Lake Erie.

Individual wells.—The gas boring at Butler is reported by Dr. Dryer to have the following section:

	Feet.
1. Hardpan (till)	15
2. Gravel and coarse sand	275
3. Red quicksand	40
4. Compact clay	45
5. Cobblestones and bowlders	3
6. Black shale	108
7. Limestone	1,064
8. Shale	500
9. Trenton limestone penetrated	89
Total	2,139

Water in large amount is found in the glacial drift. Salt water was struck in the Trenton.

The water wells in the vicinity of Butler usually obtain their supply from depths of 20 to 35 feet, though they are occasionally put down to a depth of 150 feet. In the northern part of the county, 6 or 8 miles northwest from Butler, wells are through till to a depth of 60 or 80 feet before striking water-bearing gravel or sand. The record of a well 3 miles south of Hudson was also obtained, which penetrated 75 to 80 feet of till.

At Waterloo the drift at the gas well is as follows:

	Feet.
1. Till	40
2. Sand and gravel with water	270
3. Blue clay without pebbles	45
4. Gravel and cobble	10
Total	365

At Auburn two gas wells each penetrate about 280 feet of drift, which is largely till in its upper half and largely sand and gravel in lower half.

There are occasional shallow flowing wells in the vicinity of Auburn on low ground on the border of Cedar Creek, water being obtained *from the gravel* below the till. The head is seldom more than 2 to 5 *feet below the* surface in that locality.

The gas boring at Garrett penetrates about 50 feet of till, beneath hich is 200 feet or more of sand, with a thin bed of gravel at the ottom of the drift. This sand and gravel afford a large amount of ater. In Garrett water is often obtained at 40 feet, but the wells re in some cases 75 to 100 feet. North and west from Garrett there re extensive marshes with sandy borders, where wells are obtained t slight depth without entering the till.

Along Cedar Creek Valley, in the southern part of Dekalb County, vater is usually obtained at 30 or 40 feet. The wells penetrate more gravel than till, though both are present.

A well in the southeastern part of the county, near the village of St. Joseph, and but a few rods from the bank of the river, 110 feet in depth, is mainly through sand and gravel.

A well on the farm of Christian Hirsh, east of St. Joseph River, near Spencerville, has the following section:

	Feet.
1. Yellow till	10
2. Blue till	59
3. Fine sand	120
4. Gravel	9
Total	198

In that neighborhood there are perhaps a dozen other wells 60 to 100 feet in depth, mainly through till.

NOBLE COUNTY.

General statement.—Noble County is situated immediately east of Dekalb and has an area of 420 square miles. The entire county has a morainic topography, the greater part of it being the interlobate moraine of the Erie and Saginaw lobes. In the northwestern part of the county the Saginaw moraines branch off from the interlobate belt, while in the southeastern part an Erie moraine is parallel to and closely associated with the interlobate belt.

The portion of the county occupied by the interlobate moraine includes many lakes, some of which have an area of 1 to 2 miles or more each. The most important ones are Crooked, Tippecanoe, and Goon lakes, in the southern part of the country, which constitute the head waters of Tippecanoe River. There are several in the northern part of the county and in the adjacent portion of Lagrange County, constituting the head waters of Elkhart River. A few in the western part of the county are also tributary to Elkhart River. These lakes are usually bordered by extensive marshes, though in a few cases morainic knolls and ridges rise from their immediate borders. Those in the southern part of the county tributary to Tippecanoe River have less marshy borders than the lakes tributary to the Elkhart, and the morainic hills in that portion of the county also rise more prominently above the lake basins than in the northern or western portion of the county. While the beauty of scenery does not equal

that around the lakes of Steuben County, the quality of the water is excellent and the lakes are well stocked with fish.

Except in the southeastern portion of the county wells are usually obtained at moderate depths, seldom exceeding 50 feet. The interlobate moraine and the Saginaw moraines connected with it contain usually a large amount of sand and gravel with the till. The Erie moraine, on the southeast border of the county, has much less sand and gravel associated with it, and in consequence many wells need to be sunk to a depth of 100 feet or more.

The thickness of the drift is known at three points, Ligonier, Kendallville, and Albion. At Kendallville it is 485 feet and at Albion 375 feet, but at Ligonier it is only 169 feet. It seems probable that the average thickness of the drift for the county may equal that at Albion.

Individual wells.—The following detailed record of the drift penetrated at Albion was kept by Prof. W. B. Van Gorder, of that city:

	Feet.
1. Yellow clay	10
2. Blue clay	10
3. Sand and gravel	115
4. Blue clay	20
5. Sand and gravel, with streaks of blue clay	52
6. Sand and gravel	81
7. Blue clay, with thin beds of sand	52
8. Gravel	5
9. Red bowlder clay	15
10. Sand	5
11. Slate (?)	1
12. Sand	9
Total	375

The water wells at Albion are usually obtained at 60 to 100 feet in the sand and gravel below the upper sheet of till. They penetrate from 15 to 30 feet of till at the surface. The underlying sand is either too fine or contains too little water in its upper portion to supply the wells.

The drift at the Kendallville gas-well boring (485 feet), specimens of which were preserved in a glass tube, appears to be mainly sand and gravel below a depth of 20 feet. A record of a water well in this village near the gas well was furnished by the driller, J. Hart, and is as follows:

	Feet.
1. Yellow till	10
2. Dry sand	15
3. Sand, with some gravel, but little water	50
4. Soft blue till	29
5. Gravel, with large amount of water	4
Total	108

A still deeper well at Dr. Mayer's residence in Kendallville is reported by the driller, Mr. Diebolt, to have the following section:

	Feet.
1. Surface clay	5
2. Dry gravel	20
3. Water gravel	10
4. Soft blue clay	25
5. Water gravel	20
6. Hard blue till	70
7. Soft blue till	25
8. Water gravel	10
Total	185

A well at Mr. Henry Mayer's in the east part of Kendallville, 60 feet in depth, penetrates—

	Feet.
1. Yellow till	28
2. Blue quicksand	26
3. Ferruginous crust	1
4. Water gravel	5
Total	60

From these sections it appears that the drift may have considerable variability within the limits of the village.

A well about 3 miles east of Kendallville, in section 1, Allen Township, penetrates—

	Feet.
1. Yellow till	14
2. Blue-gray till, with thin beds of sand or gravel	192
Total	206

This well is in the Erie drift above referred to.

A well 4 miles northeast of Kendallville is reported to penetrate 120 feet of till, and one in the extreme northeast corner of the county 75 feet, before obtaining water.

It is reported by Mr. Diebolt that the blue till of the Erie drift found south from Kendallville is much harder to penetrate than the blue till of the Saginaw drift found north and west of that village. A well 1 mile south from Kendallville penetrates—

	Feet.
1. Yellow till	20
2. Hard blue till	40
3. Cemented gravel	15
4. Water gravel	10
Total	85

A well in section 17, Allen Township, on a prominent drift knoll, is 177 feet deep, mainly in till.

Wells in the vicinity of Avilla, and for 5 or 6 miles west from that village, are frequently put down to a depth of 90 or 100 feet, mainly through till. Several wells in the southeastern part of the county,

in Swan and Greene townships, are reported to be mainly through till to a depth of 75 to 160 feet. Dug wells, however, are often obtained in that region at a depth of 20 to 40 feet from thin beds of sand or gravel included in the till.

In the district southeast and south from Albion records of several deep wells were obtained. One near Noblesville, 161 feet in depth, was entirely in till. Another in section 11, Greene Township, was in till to a depth of 100 feet before striking water-bearing gravel. A well in section 35, Jefferson Township, enters gravel below till at 63 feet. One in section 26, Jefferson Township, penetrates till, with thin beds of sand or gravel, to a depth of 102 feet. One in section 16, Jefferson Township, enters water-bearing gravel at 70 feet.

Northeast from Albion, in the vicinity of Rome City, one well, 117 feet in depth, penetrates—

	Feet.
1. Sand	53
2. Soft, adhesive blue clay	40
3. Sand and gravel	24
Total	117

Another, 122 feet in depth, penetrates—

	Feet.
1. Yellow till	20
2. Soft blue till	45
3. Quicksand	8
4. Soft blue till	44
5. Gravel with water	5
Total	122

A well in section 23, Wayne Township, 147 feet in depth, penetrates till, with thin beds of sand and gravel, its entire depth. Another well in the northern part of the same section has a depth of 164 feet in similar material. A well in section 27, 117 feet in depth, also has a similar section.

In the vicinity of Brimfield the surface is sandy to a depth of a few feet, but wells penetrate a large amount of till. One well, a mile east of Brimfield, 60 feet in depth, is mainly in blue till.

A well near Skinner Lake, 3 miles east of Albion, penetrates—

	Feet.
1. Yellow till	12
2. Soft blue till	52
3. Sand, with water	4
Total	68

In the northwest part of Noble County, near Hawville, wells show a variable structure, some being entirely in sand and gravel to a depth of 30 or 40 feet, while others are in till to even greater depths before striking a water-bearing gravel.

Wells in the southwestern part of the county usually have 40 to 75 feet, more generally 75 feet, of till at surface before entering beds affording a strong supply of water.

From Albion west and northwest the drift appears to be largely of gravelly constitution and wells are obtained without penetrating to eat depth. At Ligonier the depth is about 35 feet. The wells at at village usually penetrate 10 to 20 feet of till before entering the ater-bearing gravel. This gravel is shown by the gas-well boring to 125 feet in depth. There was a large supply of fresh water at out 200 feet in the gas boring, 30 feet below the rock surface.

KOSCIUSKO COUNTY.

General statement.—Kosciusko County is situated in the middle por-on of the second tier of counties from the north boundary of the tate, being immediately south of Elkhart County and west of Noble ounty. It has an area of 556 square miles, being exceeded in area y only two other counties in the State.

The eastern and southern portions of the county are situated on the nterlobate moraine of the Saginaw and Erie lobe, except the extreme utheastern portion, which is occupied by a moraine of the Erie lobe he Mississinewa). The northwestern portion of the county, com-rising about one-fourth its area, is occupied largely by gravel plains nd by marshes. It is much lower than the moraine, and yet the vide between the Wabash and the Lake Michigan drainage systems asses through it. The general elevation of the plain is about 800 feet ove tide. The moraine stands 900 feet or more in its higher portion. Along the interlobate moraine in the eastern portion of the county ere are numerous lakes, one of which, Turkey Lake, has the dis-ction of being the largest lake in Indiana. The area is probably out 8 square miles, its length being fully 6 miles and its width a le or more. There are probably a half dozen other lakes having eas of 1 to 2 square miles each, among which may be mentioned awasee, Tippecanoe, Boydstown, Barbers, Big Eagle, Little Eagle, d Pike lakes. These lakes are bordered on one or more sides by rainic knolls and ridges, but have usually extensive marshes on other les. Turkey Lake lies on the inner border of a Saginaw moraine at leaves the interlobate moraine near the border of Noble and Kos-isko counties and passes northwestward into Elkhart County. awasee, Tippecanoe, and Boydstown lakes are situated on the outer rder of the same moraine. These lakes have a pure supply of water d are said to be well stocked with fish. A biological survey of irkey Lake is in progress under the auspices of the State University. few small lakes occur along the interlobate moraine in the southern rt of the county, among which may be mentioned Yellow Creek, aver Dam, Silver, and Rock lakes. None of these have an area of square mile. They lie in the midst of morainic knolls and ridges, ith but little marshy land on their borders.

This county is as well favored, perhaps, as any in the State in abun-nt supplies of *water for* wells at shallow depth. On the plain in e *northwestern portion* the wells rarely exceed 40 feet and are

usually but 10 to 20 feet in depth. Along the moraine, even in its most elevated portion, it is rare to find wells that exceed 40 feet in depth. The drift appears to consist more largely of sand and gravel than of till, unless it be in the extreme southeastern portion of the county occupied by the Erie moraine.

The thickness of the drift is known only in the vicinity of Warsaw, the county seat, where gas-well borings show it to be in one case 247 and in another 255 feet. A well 3 miles west of Warsaw penetrates 243 feet of drift.

Individual wells.—At Milford, on a gravel plain in the northern part of the county, the wells have a depth of about 20 feet, entirely through gravel and sand. In the vicinity of North Galveston, also on the gravel plain, wells enter till below the gravel at about 25 feet. In the vicinity of Oswego, also, the bottom of the sand and gravel is reached at 25 feet or less. Water is usually obtained at both these villages and in the intervening country near the base of the gravel. West from North Galveston the plain has a till deposit at the surface, being outside the limits of the gravel overwash. But wells usually find the water-bearing sand and gravel at 15 or 20 feet.

On Bone Prairie the wells are in some cases put down to a depth of 60 feet, largely through till, there being only a slight coating of surface gravel.

At Atwood wells range in depth from 8 feet to 40 feet, but usually obtain water below yellow till at about 20 feet.

At Etna Green wells penetrate 20 or 30 feet of till before entering water-bearing gravel.

In the eastern part of the county the wells along the Tippecanoe River and southward for a mile or two are only 10 or 12 feet in depth, largely in sand. North from the river they penetrate till to a depth of 30 to 50 feet.

On the border of Turkey Lake a well 40 feet deep is reported to have penetrated:

	Feet.
1. Gravel	12
2. Till	28
3. Water-bearing gravel at bottom.	

In the vicinity of Pierceton wells occasionally penetrate 60 feet or more of till. A tubular well 1 mile northeast of Pierceton penetrates till about 100 feet.

At Kosciusko there is a surface gravel several feet in depth, but wells usually pass through it and an underlying till into a deeper gravel at about 25 feet.

For 2 to 4 miles east from Warsaw the wells are largely through till and have a depth of 18 to 40 feet.

In the east part of Warsaw a flowing well has been obtained on low ground between Pike and Center lakes at a depth of 102 feet. Its *head is about 4 feet* above the surface. At one of the gas-well borings

in Warsaw there is also a flow of water discharging rapidly from a 7-inch pipe at a height of 2 feet above the surface. As it contains much sulphureted hydrogen, it is probable that a portion of the water is from the rock strata. The overflow is probably caused from accession of water from the drift. In both the gas-well borings at Warsaw the drift is largely sand, filled with water.

In the western portion of the county, south from the Tippecanoe River, in the vicinity of Claypool, Burkett, and Mentone, wells usually penetrate 25 to 40 feet or more of till and occasionally 60 feet.

MARSHALL COUNTY.

General statement.—Marshall County is situated west of Kosciusko County, in the middle portion of the second tier of counties from the north boundary of the State. It has an area of 440 square miles.

The prominent Maxinkuckee moraine leads through the western range of townships from south to north. It also swings southeastward near the southern boundary of the county and has its inner border within the limits of Marshall County as far east as the Tippecanoe River. This moraine constitutes the principal topographic feature of the county, the remainder of the county being a gently undulating plain, with occasional slight ridging of the drift.

Lake Maxinkuckee, from which the moraine takes its name, is situated in the southwestern township of the county and is surrounded by a series of morainic knolls and ridges, which add to the attractiveness of its scenery. The lake occupies perhaps 2 square miles, and its maximum depth is 76 feet. A large number of springs occur on its borders, and its water is remarkably clear and pure. The lake is said to abound in fish. On the borders of the lake several clubhouses have been built by Indianapolis, Peru, Plymouth, and other clubs, and summer cottages are numerous along its shore.

Along the border of the lake several flowing wells have been obtained, which rise to a height of 12 to 30 feet above the lake surface. The first well driven was only 13 feet in depth. Several have a depth of but 20 or 25 feet. Others are put down to a depth of 50 to 75 feet. One well has a depth of 160 feet, and one reached a depth of 203 feet. There appear to be several water horizons, but the head is no greater from the deep wells than from the shallow ones, and the upper horizon is as strong as any.

Two other prominent localities for obtaining flowing wells are found in this county, one being along Yellow River, in the vicinity of Plymouth, the other along Yellowbank Creek, in the vicinity of Teegarden. Those in the vicinity of Plymouth are usually but 40 or 50 feet. Those in the vicinity of Teegarden are 40 to 100 feet. In all cases the flowing wells are from beds of sand or gravel beneath a sheet of till.

The *bottom of the drift* has, so far as known to the writer, been

reached in but one locality, Plymouth, the county seat, where a gas boring entered rock at 242 feet.

Individual wells.—On the head waters of Yellow River, near Bremen, in the northeastern part of the county, wells are usually but 12 to 20 feet in depth and penetrate mainly sand and gravel.

At Bremen, however, there is a small ridge of till on which the average depth of wells is about 40 feet. One well in the west part of the village is 100 feet in depth, mainly blue till.

Wells at Inwood are mainly through sand and gravel, though penetrating thin beds of clay; the depth ranges from 20 to 50 feet. Between Inwood and Plymouth the usual depth is about 35 feet, water being obtained in sand or gravel below till.

The gas well at Plymouth has the following section of drift:

	Feet.
1. Sand and gravel	40
2. Till	50
3. Mainly sand and gravel	150
Total	240

Flowing wells in the vicinity of Plymouth are obtained in a narrow belt, scarcely 40 rods in width, along the Yellow River bottoms. They usually penetrate several feet of sand at the surface, below which are alternations of till with sand beds. The best flow is obtained at a depth of 42 to 45 feet.

The flowing wells along Yellowbank Creek are found for 2 or 3 miles west from Teegarden. They penetrate much blue till before reaching a water vein. The water horizon appears to vary greatly in depth in the several wells, there being a range from 40 feet to about 100 feet.

In the southern part of the county, on the plain east and north from the Maxinkuckee moraine, water is usually obtained at 10 to 20 feet. At Tippecanoe Station, on the borders of Tippecanoe River, the depth is but 10 or 15 feet. In the north part of Walnut Township, where the surface is sandy and poorly drained, wells seldom exceed 10 feet in depth. In portions of Walnut Township, although surface bowlders abound as in till tracts, yet the underlying deposits are mainly sand, and wells are found at depths of 10 to 20 feet.

The deepest of the flowing wells at Lake Maxinkuckee is at the residence of D. W. Morman. At the time of the writer's visit the water scarcely reached the surface, 20 feet above the level of the lake. Of the 203 feet penetrated, fully 90 per cent is thought to have been till, the sand beds being but a few feet in thickness.

Messrs. Thompson and Lee, in their report on Marshall County,[1] note two wells on the northeast shore which have a head 31 feet above

[1] Fifteenth Ann. Rept. Indiana Geol. Survey, pp. 182-186.

the surface of the lake. The wells are each 72 feet in depth and have the following section:

	Feet.
1. Soil and yellow clay	8
2. Sand	14
3. Blue clay	38
4. Sand and gravel	12
Total	72

A short distance east from these wells the head in a well 50 feet in depth is but 19 feet above the lake. The well at the Peru Clubhouse, on the east side of the lake, was bored to a depth of 160 feet and obtained only a weak flow. At the Indianapolis Clubhouse a good flow was obtained at only 27 feet. At the Highland House a well 33 feet in depth entered water-bearing sand at a depth of 13 feet. Near the Highland House D. W. Morman, of Indianapolis, has several wells. Four of them, averaging about 22 feet in depth, are estimated to have a combined discharge of 15 barrels per minute. These wells feed a ram which supply the grounds with water. Mr. Morman also has a deeper well, with the following section, in which the flow is obtained from the sand above the blue clay:

	Feet.
1. Yellow clay	11
2. Sand	25
3. Blue clay	62
Total	98

STARKE COUNTY.

General statement.—Starke County is situated on the eastern border of the head of the Kankakee marsh, in the northwestern part of the State. Its area is about 300 square miles. It is a much lower district than Marshall County on the east, and the surface is covered by sand dunes or marshes, except in the southeast portion, where a small till plain occurs which is nearly free from sand. Underneath the sand, throughout the inhabited portion of the county, a blue till is often found at depths of 10 to 40 feet, but wells are usually obtained in the sand above the till.

A boring at North Judson, for the purpose of obtaining natural gas, shows the drift to have a thickness of 198 feet. A gas boring at Knox also penetrated a large amount of drift, but the precise depth at which rock is struck has not been ascertained. In both borings till is entered below the sand at a slight depth and constitutes the main part of the drift.

Individual wells.—At Knox, the county seat, wells are obtained at various depths, ranging from 14 up to 40 feet, the deeper ones being on sand ridges and the shallow ones on low ground among the ridges. At the gas-well boring at Knox the water rose nearly to the level of

the surface, but the pipes have been withdrawn and the well abandoned. Some of the shallower wells are very liable to contamination.

At North Judson the water is obtained near the base of the sand at depths of 16 to 30 feet, the deeper wells being on sand ridges. There appears to be a liability to contamination of water in this village.

A boring at the north end of Cedar Lake, in the southeastern part of the county, is reported to have the following section:

	Feet.
1. Sand	5
2. Blue clay	9½
3. Sand and gravel	3
4. Blue clay	18
Total	35½

Cedar Lake is said to have a depth of 34 feet, with bowlder clay at bottom.[1]

Near the east border of the county several wells have been put down to depths of 30 to 100 feet, mainly through till. The deepest one (at Mr. Pette's, in sec. 24, T. 32, R. 1 W.), about 100 feet in depth, has the following section:

	Feet.
1. Yellow till	10 to 12
2. Blue till	16
3. Gray sand	52
4. Blue clay	4
5. Cemented sand and gravel	15
Total	97 to 99

A mile north from Mr. Pette's, at David Fetter's, a well penetrates—

	Feet.
1. Sandy till	5
2. Blue till	13
3. Quicksand	5
4. Blue till	30
Total	53

NEWTON COUNTY.

General statement.—Newton County is situated on the west border of the State, immediately south of Kankakee River. It has an area of about 400 square miles. The northern third of the county falls within the limits of the Kankakee marsh and the sandy ridges which form the southern border. The central portion of the county is occupied by a moraine 3 to 5 miles in width, which leads across it from west to east. This moraine rises from 50 to 75 feet above the bordering plains on either side and has a gently undulating surface. South from the moraine is a plain traversed by the Iroquois River, which extends southward beyond the limits of the county.

In the sandy northern portion of the county wells were, in the early days of settlement, put down to a slight depth and obtained their *water in the sand*; but within the past few years they have been

[1] Fifteenth Ann. Rept. Indiana Geol. Survey, p. 22A.

nk to depths of 50 to 100 feet or more, through a sheet of blue till ich underlies the sand, to beds of sand and gravel which furnish a tter quality of water than that in the surface sand. Along the rainic ridge wells are usually obtained in the glacial drift, but ve occasionally been put down in the underlying rock. The disnce to rock is 100 to 150 feet or more. The best wells on the raine are deep ones, extending nearly to the base of the drift. e moraine appears to be composed of a nearly solid bed of till. In e plain south of the moraine many wells are sunk 75 to 100 feet or ore to obtain a better supply of water than is afforded at shallower pths. On the immediate borders of the Iroquois River wells reach depth of 100 feet without entering rock, but in the southern portion the county rock is often entered at 25 feet or less. The rock rface appears to be generally much higher on the southern borders the county than in the district north from the Iroquois River.

Individual wells.—Records of several deep wells in this county are resented in the Twelfth Annual Report of the State Geologist, and re also given below:

Twelve wells, bored by Hyer Brothers, in and near Kentland, penrated, on an average, the beds named below:

	Feet.
Soil	2
Yellow clay	10
Blue glacial clay	45
Sharp sand	2
Total	59

Water rises to within 18 or 20 feet of the surface.

The well at the public square in Kentland obtained a flow of water 300 feet and penetrated the following strata:

	Feet.
Glacial deposits	150
Black slate	73
Devonian limestone	25
Upper Silurian limestone	120
Total	368

At Kent's warehouse, in Kentland, an unsuccessful boring for water netrated:

	Feet.
Glacial deposits	82
Black slate	80
Total	162

A well at Alexander Kent's, 1 mile northeast of Kentland, 320 feet depth, penetrated 182 feet of drift. A well on his farm, 4 miles rtheast of Kentland, found only 50 feet of glacial drift and obtained ter in limestone at 53 feet. A well on another farm, 2 miles southst of Kentland, struck limestone at 50 feet and there obtained ter.

At Mr. Brush's, 3 miles southwest of Kentland, a flowing well has the following section:

	Feet.
1. Soil and yellow clay	9
2. Sand and gravel	4
3. Blue clay	68
4. Hardpan gravel	4
5. Limestone	1
Total	86

Another flowing well was obtained at Mr. Drake's, sec. 25, T. 27, R. 9 W., 3 miles east of Kentland. After penetrating 33 feet of till it enters a bed of sand and gravel, from which the flow is obtained.

Several flowing wells have been obtained in the Iroquois Valley on the overflow plain of the river. An average section of several of these wells, furnished by David McKenzie, their driller, is as follows:

	Feet.
1. Soil and sand	14 to 17
2. Blue clay	30
3. Hardpan gravel	5
4. Blue sand with flow of water	4
Total	53 to 56

A well near the State line west of Kentland, reported by Mr. McKenzie, penetrates—

	Feet.
1. Soil	3
2. Yellow clay	6
3. Blue glacial clay	25
4. Blue glacial clay with partings of sand	46
5. Gravel and fine sand	3
Total	83

A well on Mr. Peterson's farm, near the State line, in sec. 11, T. 28, R. 10 W., on a high part of the moraine, referred to above, penetrates 168 feet of drift without reaching the rock. Mr. McKenzie reports the following section:

	Feet.
1. Soil and yellow clay	6
2. Blue bowlder clay	154
3. Fine gravel and sand with water	8
Total	168

An almost identical series was found at Mr. Besicker's, in the adjoining section on the south, where a well 171 feet in depth was in till to a depth of 165 feet before striking water-bearing strata.

About 5 miles northwest from Kentland, in sec. 13, T. 27, R. 10 W., four wells on the farm of Mr. J. V. Speck are reported to have passed through a brown swamp muck at the bottom of the till, about 80 feet below the surface, beneath which there was water-bearing sand and gravel.

At Morocco a boring at the Creamery penetrated 127 feet of till, at hich depth limestone was entered.

Several wells near the range line east of Morocco, on the north ope of the moraine, enter rock at 110 to 120 feet, after penetrating solid bed of till.

At Mount Ayer, also on the moraine, near the eastern border of the unty, many wells get a good supply of water in gravel at 30 or 35 et, but a few have been put down to depths of 140 to 160 feet. Mr. shby's well penetrated 120 feet of drift and 20 feet of rock, as follows:

	Feet.
Yellow till	10
Blue sand	20
Blue till	90
Blue shale	20
Hard flint rock at bottom.	
Total	140

Mr. Lewis Marion's well in the southwest part of the village strikes hale at 139 feet. W. J. Young's well strikes shale at about 150 feet, nd a hard rock at 162 feet. The drift is mainly till.

In the vicinity of Brooke a soft till is penetrated to a depth of 50 or 0 feet, below which there is a harder till, extending either to the rock r to water-bearing beds. Several wells between Brooke and the east ne of the county overflow when on the low bottoms of the Iroquois liver, and rise nearly to the surface on the plain that borders the iver. Their depth ranges from 80 feet up to 120 feet or more. Rock usually struck at 100 to 120 feet. The following section at J. B. yon's, 1 mile north of Brooke, will illustrate the structure:

	Feet.
Yellow till	10
Blue sand, very fine	5
Soft blue till	55
Blue-brown till	36
Total	106

Rock was struck at 106 feet. Water stands 12 feet below the surface.

In the southwest part of T. 27, R. 8 W., and southeast part of 27, R. 9 W., near the south line of the county, several wells strike ck at 40 or 50 feet or less, and there are occasional outcrops of the ockport (often called Niagara) limestone in low knolls rising a few et above the level of the bordering plain. Eastward from there to oodland the drift appears to be generally thin. Rock is struck at out 20 feet. The wells which obtain their water from the Lockport nestone, in the southern part of the county, often reach a depth of 0 to 300 feet, and obtain a water charged with sulphureted hydrogen. A natural gas boring at Kentland, 1,325 feet in depth, obtained a lphurous water with head 45 feet below the surface. It supplies the aterworks.

JASPER COUNTY.

General statement.—Jasper County is situated in the northwestern part of the State, immediately east of Newton County and south of the Kankakee marsh. Its area is 570 square miles, the second county in area in the State. Like Newton County, its northern third is covered by the Kankakee marsh and by sand ridges that form the south border of the marsh. Its central portion is traversed by a continuation of the moraine that passes eastward across Newton County. Its southern portion is a plain covered quite extensively with sand, which in places is drifted into dunes.

In the northern part of the county the drift deposits are 75 to 200 feet in depth, but in the central and southern portions, although the altitude is as great as in the northern portion, the drift deposits are generally quite thin, there being often only a coating of sand 10 to 20 feet in depth. In some wells in the northern part of the county (cited below) a hard till is found below the soft Wisconsin till which is probably Illinoian drift.

Individual wells.—On the borders of the Kankakee marsh, in the northern part of the county, wells are usually obtained at slight depth in the sand deposits. But upon approaching the moraine in the central portion of the county the sand has insufficient depth to afford good water, and wells are accordingly sunk to beds of sand and gravel beneath the till.

Two wells at Surrey enter rock at about 90 feet. They are mainly through till. Several borings for gas 2 or 3 miles north of Surrey, on the edge of the sandy ridges, strike rock at 80 or 90 feet. The drift is mainly till.

On the crest of the moraine southeast of Surrey a well at Charles Coen's, sec. 12, T. 29, R. 7 W., strikes rock at 192 feet, and there obtains water. A well at William Nowel's, in the same section, strikes rock at 140 feet and obtains water at 162 feet. In both wells the drift is mainly till.

In sec. 30, T. 30, R. 6 W., on the north border of the moraine, a well at Jacob Ush's obtains water at 146 feet without striking rock. Another at George Marcum's obtains water at 130 feet. In both wells a soft till is penetrated for about 90 feet, below which is a hard till, extending to the water-bearing sand and gravel. At J. S. Williams's, a short distance north from the wells just mentioned, a well was obtained at the bottom of the soft till at a depth of 97 feet.

A well in a low, marshy tract in sec. 15, T. 30, R. 6 W., enters rock at about 75 feet.

A well on the moraine in sec. 13, T. 30, R. 5 W., penetrates 102 feet of drift, mainly blue till. Another well in the same section penetrates 80 feet. A well at Mr. Osborne's, in section 1 of the same township, has 126 feet of drift, and one at Mr. Rayburn's, in section

2, 118 feet. In all these wells the water is obtained at slight depth in the underlying rock.

On the plain north of the moraine, in sec. 27, T. 31, R. 5 W., a well at James Gill's enters rock at 47 feet, after penetrating considerable till.

Passing southward near the east side of the county to the south border of the moraine, the rock is found to rise within a few feet of the surface. Wells in sec. 3, T. 29, R. 5 W., penetrate but 5 to 15 feet of drift. This region of thin drift extends southward beyond the limits of the county and westward to Rensselaer. Some of the wells obtain their water without entering the rock, but a large number are carried a few feet into the rock.

Immediately west of Rensselaer the rock surface suddenly drops down 75 or 100 feet or more, so that wells are 80 to 150 feet in drift. For example, a well on the south border of the moraine, in sec. 29, T. 29, R. 7 W., enters rock at 143 feet. A well on the plain south of the Iroquois River, in sec. 3, T. 28, R. 7 W., enters rock at about 80 feet. The rock surface continues low for several miles south from the Iroquois River, in the western part of the county, and wells are often 50 to 75 feet in depth without entering the rock.

In the southern half of T. 27, R. 7 W., the rock surface rises to a greater altitude than in the vicinity of Rensselaer. Although the general elevation is about 50 feet higher than at Rensselaer and the portion of the county east from that city, the wells in this southern end of the county often enter rock at 10 to 20 feet.

At Remington the dug wells are usually 12 to 20 feet in depth, obtaining water near the base of the drift. The drilled wells are 75 to 125 feet in depth. Robert Parker's well, 125 feet in depth, has the following section:

	Feet.
1. Drift	18
2. Shale	95 to 100
3. Solid stone, probably limestone	8 to 10
Total	125

The water rises from this depth within 18 feet of the surface. It contains sulphureted hydrogen in small amount.

Wells for water in the vicinity of Rensselaer range in depth from 30 feet to 200 feet. They enter rock at 6 to 15 feet. On the low ground along the river several of them have head sufficient to overflow, but at the general level of the town the water rises only within 5 or 6 feet of the surface. A gas-well boring at Rensselaer was carried to a depth of 1,275 feet, but the boring is not utilized, although a sulphurous water rises to the surface. This water is probably from the Lockport limestone, as in the water wells of the vicinity.

PULASKI COUNTY.

General statement.—Pulaski County is situated in the northwestern part of the State, immediately east of Jasper and south of Stark County. Its area is 430 square miles. Like Stark County, Pulaski is almost entirely occupied by sand ridges and marshes, there being only small areas of cultivable land free from sand, found chiefly on the eastern and southern borders of the county. Although it occupies the divide between the Kankakee, a tributary of the Illinois, and the Tippecanoe, a tributary of the Wabash, there is no dividing ridge, and large tracts in the northwestern part of the county might be drained with equal readiness to either stream. No adequate drainage lines have been developed in the portion of the county west from the Tippecanoe River. The portion east from the river is much better drained.

Individual wells.—In the northwestern portion of the county large areas have never been settled. On the tracts which are cultivated wells are usually obtained at 20 to 40 feet or less without reaching the bottom of the sand. In the southwestern portion of the county the drift deposits are very thin, and wells usually enter rock at 10 to 20 feet. At Medaryville, however, a well at the tile factory penetrates 92 feet of drift and obtains its water supply from the rock at 115 to 120 feet.

In the eastern part of the county wells are usually obtained at shallow depths, either in the sand or in beds of sand and gravel associated with the till.

At Winamac, the county seat, the wells often reach a depth of 50 to 60 feet. They pass through 10 to 25 feet of sand, or a sandy clay loam, below which is a bed of blue till, extending to the water-bearing gravel. The drift at this point is 110 feet in thickness. The gas-well boring, 1,200 feet in depth, obtains a flow of water estimated at 45 gallons per minute. The horizon at which the water was found has not been ascertained.

FULTON COUNTY.

General statement.—Fulton County is situated near the middle of the third tier of counties from the north boundary of the State, immediately east of Pulaski and south of Marshall County. It has an area of 380 square miles.

The surface is very diversified. In the western portion there is a till plain on which occasional drift ridges and knolls occur, and also a few sand ridges. Along the northern boundary there is the Maxinkuckee moraine, and in the eastern and southeastern portions the western slope of a bulky moraine formed by the Erie lobe, which connects in northeastern Fulton County with the Maxinkuckee moraine to form the great Erie-Saginaw interlobate moraine, which, as above noted, leads northeastward into Michigan through the northeastern

part of Indiana. West from the junction of these two moraines there is much marsh land with sandy ridges, which is imperfectly drained by southern tributaries of Tippecanoe River.

The thickness of the drift is known at Rochester, in the central portion of the county, and at Kewanna, in the western portion. At Rochester an artesian well at the court-house has 155 feet of drift, while a gas-well boring, 60 rods northeast from the court-house, has 245 feet. The altitude of the two wells is very nearly the same, and in both there are alternations of till with sand and gravel. At Kewanna the drift is 167 feet in one well and 208 feet in another. It is largely assorted material, but contains beds of blue till.

Individual wells.—Very few records of the sections of wells were obtained in this county, largely because of their shallowness. The great majority of wells are obtained at depths of 20 or 25 feet; in many cases at 12 or 14 feet. In the extreme northeast corner of the county, however, there is an area of a township or more on the east side of the Tippecanoe River where a compact till makes it necessary to sink wells to a depth of 60 or 80 feet. In one section six wells have an average depth of 70 feet. Another place in which wells are sometimes difficult to obtain is found in the northwest township of the county, in the Maxinkuckee moraine, but here it is rare for a well to exceed 60 feet in depth.

WHITLEY COUNTY.

General statement.—Whitley County is situated in the northeastern part of the State, immediately west of Allen and south of Noble County. It has an area of 330 square miles.

The extreme northwestern border of the county is occupied by the interlobate Erie-Saginaw moraine. The remaining portion of the northwest half of the county is largely occupied by the Mississinewa moraine of the Erie lobe, which closely flanks the interlobate moraine. The inner or southeastern border of the Mississinewa moraine is followed more or less closely by Eel River. The portion of the county lying east and south of Eel River is largely plane surfaced, though occasional knolls and ridges of drift give it some diversity.

This county is preeminently a clay county, there being but limited areas in which sand or gravel is found at the surface or at slight depth. The drift to a depth of 50 to 60 feet or more is usually a compact till in which only weak wells can be obtained. The most conspicuous exception is a narrow belt along Eel River, scarcely more than a mile in width, in which deposits of sand and gravel appear at the surface, and strong wells are obtained at slight depth.

The thickness of the drift is known at Columbia City, the county seat, and at Larwell, a village about 10 miles west from Columbia City. At Columbia City the gas boring made on low ground near Eel River has 224 feet of drift, mainly sand. At Larwell the gas boring

made on an elevated part of the moraine has 365 feet of drift. At this point there is about 100 feet of till at the surface, below which the drift is mainly sand and gravel. In both wells large quantities of water were obtained before reaching the rock. That at Columbia City overflows. It is probable that in the extreme southeast corner of the county the drift is less than 100 feet.

Individual wells.—But few records of wells were obtained in this county, since the sections appear to differ but little in structure. The compact till is reported in the majority of wells in every township of the county. Occasionally considerable variety is found in the distance to the water beds within the limits of a single village, but as a rule no strong wells have been obtained above the general water stratum, 50 to 75 feet below the surface.

At Churubusco, in the extreme northeast part of the county, the deepest well is reported to be 80 feet, while several are 50 feet or more.

At Collins, 4 miles southwest of Churubusco, several wells are 75 or 80 feet, through till.

At Columbia City, which is situated near the Eel River valley, wells often reach the bottom of the till at 30 or 40 feet.

The deepest water well of which record was obtained is on the plain in the southeastern part of the county, and penetrates—

	Feet.
1. Till	67
2. Sand and gravel, with beds of till	130
3. Water gravel	5
Total	202

ALLEN COUNTY.

General statement.—Allen County is situated in the northeastern part of Indiana, on the east border. Fort Wayne is its county seat. It has the distinction of being the largest county in the State, having an area of 650 square miles.

The greater part of the county has a level surface, though there are two well-defined morainic belts which pass through it. The St. Marys moraine enters in the southeast part and follows the northeast border of the St. Marys River to Fort Wayne. It then finds its continuation northward on the east side of the St. Joseph River. The Wabash moraine enters the county near Zanesville, in its southwest corner, and passes thence northward to the old lake outlet near Aboit. North from the outlet its course is northeastward along the northwest side of the St. Joseph River. There is a narrow plain in southern Allen County between these two moraines, but in northern Allen County they are separated only by the valley of the St. Joseph River. A plain in the northwestern part of the county extends beyond its limits into Dekalb and Whitley counties. The plain in the eastern part of

the county, formerly covered by the glacial lake which discharged southwestward to the Wabash, has a very smooth surface, with only an occasional low ridge of sand or bar of gravel formed by the lake waters.

In the southern half of the county the thickness of the drift is very much less than in the northern half, though probably averaging not less than 75 feet. At Fort Wayne it is 60 to 125 feet or more. Near the north border of the county the drift exceeds 200 feet, two borings having reached that depth without entering rock. In the south part of the county it is in places but 20 or 30 feet.

In this county, as in Whitley, the greater part of the drift penetrated by wells is a compact till. Wells are, as a rule, more easily obtained than in Whitley County. Along the St. Joseph River there is a narrow belt of gravelly land. There is also a gravel plain of some extent in the northwest part of the county, near Huntertown, which connects the gravel deposits of the Cedar Creek valley with the gravel belt on Eel River, thus crossing the continental divide between the St. Lawrence and the Mississippi basins. It is needless to say that there is no ridge along this part of the divide.

Individual wells.—A well at the court-house in Fort Wayne, sunk in 1875 for artesian water, reached a depth of 3,000 feet, but its head fell short several feet of coming to the surface, 772 feet above tide. The drift at this boring is 88 feet. A boring for natural gas in the old lake outlet, north of the St. Marys River in Fort Wayne, at an elevation about 750 feet above tide, penetrates 96 feet of drift. One 3 miles west, also in the lake outlet, has 90 feet of drift. The waterworks of Fort Wayne are supplied from about fifty wells, some being in gravel at depths of 40 to 60 feet, others in rock at 250 to 450 feet. Abbott's gas-well boring in the eastern part of the city, at an elevation nearly 800 feet above tide, penetrates 106 feet of drift. In this, as well as several other borings in the vicinity of Fort Wayne, the lower portion of the drift is found to contain much sand and gravel. In one boring near the bank of the Maumee River Dr. Dryer reports a flowing well, depth not given, which upon a partial analysis shows hard water, with a trace of sodium chloride and a small amount of sulphureted hydrogen; also iron carbonate in measurable amount.[1]

Wells near the county infirmary, southwest of Fort Wayne, on the border of the lake outlet, obtain water at 25 or 35 feet. After penetrating a few feet of surface sand they are mainly through till. In a few cases the wells reach a depth of 60 feet.

In the portion of the county north of the lake outlet and west of the St. Joseph River there is usually but 20 or 30 feet of till to be penetrated before a strong stream of water is found in sand or gravel. Possibly a continuous sheet of gravel underlies that region at comparatively slight depth. In one case a boring near the north line of

[1] *Sixteenth Ann. Rept.* Indiana Geol. Survey, p. 127.

the county, after penetrating 20 feet of surface till, was carried 125 feet into sand without reaching its bottom.

Reference was made in the discussion of Dekalb County to several wells near Spencerville which have a depth of 60 to 100 feet. Some of these are located in northern Allen County. The well at Christian Hirsh's, 198 feet deep, is very near the county line. This well, after penetrating 70 feet of till, passed through 120 feet of fine sand before entering a material coarse enough to screen. Sand of such fineness is frequently a greater obstacle to well drillers than a solid blue till free from water.

Deep wells are more common in the northeastern part of the county than elsewhere, there being a heavy sheet of till on the moraine east of the St. Joseph River and also on the old lake plain. It is not rare to penetrate till 60 or 75 feet before obtaining water.

From Fort Wayne eastward to New Haven rock is usually encountered at 80 feet or less. In some cases wells are obtained at 25 or 35 feet in beds of gravel between the sheets of till. There are also places where wells may be obtained in the sand of the old lake outlet at depths of 20 feet or less.

At the village of New Haven the wells are obtained in gravel at a depth of about 30 feet. There is, however, a well at E. W. Green's, just east of the village, situated on the upper lake beach, which has the following section:

	Feet.
1. Beach sand and gravel	9
2. Blue clay	33
3. Gravel	3
4. Blue till	23
5. Cobble and gravel	5
Total	73

Another well at Mr. Green's, 55 feet in depth, penetrates 18 feet of surface sand, and is then through blue till to the bottom. There are two wells at Herman Schuelker's, in New Haven, each about 60 feet in depth, which are mainly till. The waterworks supply at New Haven is from a gas boring 300 feet in depth. In this boring rock was struck at 82 feet. At about 150 feet a strong flow of water was obtained, which rose nearly to the surface, and at 300 feet the head was increased to 11 feet above the surface. An analysis of the water is given elsewhere.[1]

A well at Robert Bell's, east from New Haven, near the State line, strikes rock at 60 feet, and obtains water at that depth.

The southeastern portion of the county usually affords strong veins of water at convenient depths, seldom more than 30 or 40 feet. The drift in that portion of the county is probably not more than 75 feet in average thickness.

[1] Eighteenth Ann. Rept. U. S. Geol. Survey, Part IV, p. 498.

In the southwestern and southern portions of the county records of a few wells were obtained which penetrate 20 or 30 feet of till before reaching a water gravel. Occasionally a well is sunk to a depth of 75 feet, largely through till. One well on the moraine in the southwest corner of the county, 102 feet in depth, is mainly through till. Another, 85 feet in depth, is also largely through till. Upon passing eastward from the moraine in the southwest township of the county tubular wells are found to decrease in depth, being 75 or 100 feet on the moraine and but 40 or 50 feet on the bordering plain.

ADAMS COUNTY.

General statement.—Adams County is situated on the east border of the State, immediately south of Allen. Its area is 330 square miles. Like Allen County, it has a large amount of very plane surface, whose monotony is broken by two moraines, the St. Marys and the Wabash. The St. Marys moraine crosses the northeast corner of the county, and occupies a belt 2 to 4 miles in width along the east border of the St. Marys River. The Wabash moraine traverses the southern and southwestern portion of the county, following the northeast border of the Wabash River, and has a breadth seldom exceeding 2 miles. The altitude of the southern portion of the county is somewhat higher than that of the northern portion, but the general descent in the 24 miles from south to north across the county amounts to scarcely 100 feet.

The moraines and also the plain tracts present a nearly unbroken sheet of till. There appear to be no definite gravel aprons or outwash deposits from the moraines. In places where the drift is thick considerable sand and gravel is found in its lower portion, but as a rule the drift deposits are thin, their average thickness being scarcely more than 50 feet, if the deep preglacial valleys are disregarded. Wells often reach rock at but 30 or 40 feet or less, and the shallow valleys of the St. Marys and Wabash, scarcely more than 30 feet in depth, have numerous rock outcrops along their courses in this county. The great depth of valley excavation prior to the drift deposition is shown by the gas-well boring at Geneva, in the southern part of the county, where 350 feet of glacial drift was encountered. Within a mile of the boring at Geneva rock outcrops occur at an elevation as high as the well mouth. Notwithstanding the great depth of these preglacial valleys, they are so completely filled that no surface indications of their courses can be seen.

Individual wells.—Along the St. Marys moraine and the plain east of it, in the northeastern part of the county, the depth of wells ranges from 20 to 50 feet, mainly through till.

At Decatur rock is struck at the court-house well at 40 feet. It penetrates till, with the exception of a few feet of sand at the bottom. Several wells in the city obtain their water supply from this bed of

sand and gravel above the rock. The city waterworks obtain a supply from limestone at 250 feet.

At Pleasant Mills, near the east border of the county, wells are 20 to 35 feet in depth, mainly through till. They usually obtain water in a bed of sand or gravel, but a few enter the rock.

On the plain south and west from the St. Marys River records of several wells were obtained which enter rock at 50 or 60 feet. They usually penetrate from 35 to 50 feet of till, beneath which a bed of sand is found, which, if not of too fine texture, is made the source of water supply, but if very fine it is cased out, and water is obtained from the underlying rock. On portions of this plain rock is encountered at 20 to 25 feet.

On the Wabash moraine, in the southern part of the county, records of two wells were obtained in sec. 32, T. 25, R. 15 E., which enter rock at 51 and 60 feet. They each penetrate 35 or 40 feet of till, below which is sand. A well on the moraine in section 31 of the same township enters rock at 70 feet. It penetrates till 52 feet, below which is sand and gravel.

Along the Wabash River wells are usually obtained at 15 or 20 feet, either at the surface of the rock or at slight depth in it.

At Geneva, which is situated near the Wabash River, the wells usually obtain water in gravel below till at about 35 feet. A few are sunk into limestone and have a depth of 80 feet. The gas-well boring, as above noted, penetrated 350 feet of drift. Of this, the upper 80 feet was principally till; the remaining 270 feet was mainly sand and gravel.

On the plain southwest from the Wabash River rock is usually encountered at but 20 or 30 feet. The majority of wells, however, are obtained without penetrating the rock.

WELLS COUNTY.

General statement.—Wells County is situated immediately west of Adams and south of Allen County, in the northeastern part of the State. It has an area of 357 square miles. Its topography is quite similar to that of Adams County, there being a plain surface except in two moraines, one of which crosses the northeastern portion of the county and the other the southwestern. The one in the northeastern part is the continuation of the Wabash moraine of Adams County. It follows the northeastern border of Wabash River to about the middle of the county, where it turns north away from the river and traverses Allen County, as noted above. The moraine which crosses the southwestern corner follows the northeast border of Salamonie River and receives the name Salamonie from that stream. Neither of these moraines much exceed 2 miles in average width in their course through this county. There is also a slight ridging of the drift along

the northeast border of Rock Creek, parallel to and midway between the two moraines just mentioned. Its relief is, however, but 15 or 20 feet, or scarcely half as great as the relief of either of the moraines mentioned. The streams of this county, like those of Adams, are flowing in small, shallow valleys, cut but 20 to 40 feet into the plains.

The drift is very similar to that of Adams County, being a somewhat uniform sheet of compact till at surface, with thin beds of sand included in or underlying it. The thickness of the drift is also moderate, rock being struck at many places at 30 to 50 feet, and occasionally at less depths. The average thickness for the county, unless greatly affected by concealed deep valleys, probably does not exceed 50 feet. There are no surface indications of deep valleys traversing the county, nor have borings shown their presence. The deep borings, however, are not sufficiently numerous to afford much knowledge of the topography of the rock surface.

Individual wells.—Wells along the Wabash moraine, in the north part of the county, usually penetrate but 50 or 75 feet of till, and tubular wells are usually put down to these depths. The dug wells obtain moderate amounts of water from local beds of gravel or sand in the till at much less depth. At Kingsland, on the crest of the moraine, a record of one well was obtained, which enters rock at 80 feet, but another well of the same depth does not reach rock; both are mainly through till.

West of the Wabash moraine rock is usually found at 50 to 70 feet, while in the vicinity of Uniondale it is struck at 20 to 40 feet. The tubular wells often enter the rock. Dug wells usually obtain supplies of water in the lower portion of the drift.

Near the eastern border of the county the drift on the moraine is about 100 feet in thickness, and several wells in that vicinity are 60 to 100 feet in depth. On the plain north of the moraine wells are usually obtained at less than 50 feet.

Along the Wabash River there are numerous outcrops of rock, but wells are ordinarily obtained above the rock at depths of 20 feet or less.

At the city of Bluffton, the county seat, the supply for the waterworks is obtained from wells drilled into the rock to a distance of 150 to 300 feet.

On the plain southwest from the Wabash River wells are usually obtained at 40 feet or less in beds of sand or gravel below the till. Occasionally a well is sunk to a depth of 60 or 75 feet.

On the Salamonie moraine the wells range from 20 feet up to 75 feet or more. They are principally through till.

Along the Salamonie River, and on the plain west of the river, wells are seldom more than 30 feet in depth, and are usually through till to the water vein near the bottom.

HUNTINGTON COUNTY.

General statement.—Huntington County is situated in northeastern Indiana, west of Wells and the southern portion of Allen County, and has an area of 380 square miles. Huntington is its county seat. The greater part of this county is a smooth, sloping plain, standing 800 to 900 feet above tide. The Salamonie moraine, which enters it from Wells County in the southeastern corner, becomes scarcely traceable near the center of the county, and no other moraine of prominence is found within the limits of the county. The valleys are small, with one exception, that of the lake outlet, which joins the Wabash at Huntington. This outlet has a depth of 40 to 75 feet and a breadth of 1 to 2 miles throughout its course in the county, whereas the Wabash, above its junction with the outlet, and also the Salamonie, have valleys scarcely one-eighth of a mile in average width and of less depth than the lake outlet.

The elevation of the rock surface decreases gradually northward from the southern boundary of the county to the lake outlet. North of the lake outlet it appears to drop down rapidly. The drift is comparatively thin in the portion of the county south of the outlet, there being seldom so much as 100 feet encountered in wells. North from the outlet the thickness of drift is 150 to 200 feet or more.

In Huntington County, as in Wells and Adams, the drift is mainly a compact till. Wells are obtained either in local deposits of gravel and sand within the till or more extensive deposits which appear to prevail near the base of the drift.

Individual wells.—Wells on the plain north of the Wabash outlet have, in a few cases, entered rock at about 140 feet after penetrating a nearly solid sheet of till. Ordinarily, the wells are but 50 to 100 feet and encounter no rock.

In the city of Huntington, on the uplands bordering the lake outlet, wells are frequently obtained at about 30 feet in gravel below till. Along the outlet they are usually sunk into the limestone, which there outcrops quite extensively. The waterworks wells are sunk into the limestone to a depth of about 100 feet. Although there are 11 wells in use, the supply is insufficient and some water is pumped from the river.

Above Huntington, in the vicinity of Roanoke, wells in the old lake outlet are often obtained without entering rock, at depths of 10 to 25 feet. One well near Roanoke enters rock at 36 feet.

In the vicinity of New Lancaster, on the borders of the Salamonie River, gravel is more abundant than elsewhere along the valley and wells often obtain water without penetrating till. Between New Lancaster and Andrews wells commonly penetrate 20 to 40 feet of till.

At Andrews, the wells are usually carried into the limestone to *depths of 60 to 150* feet, and occasionally to over 200 feet. The analy-

sis of water from a well in this village, 214 feet in depth, is given elsewhere.[1] The water is shown to be slightly saline as well as hard. It also contains much sulphureted hydrogen.

In the vicinity of Warren several flowing wells have been obtained in limestone on low ground along the Salamonie River at a depth of 65 to 100 feet. The head is sufficient to carry the water only 5 or 6 feet above the surface. The water is decidedly chalybeate.

WABASH COUNTY.

General statement.—Wabash County is situated in the north-central portion of the State, with the city of Wabash as its county seat. Its area is 430 square miles. The eastern border of the county is occupied throughout its entire length by the Mississinewa moraine. The northwestern part of the county is occupied by the Erie-Saginaw interlobate moraine, the southeast border of that moraine being near the valley of Eel River. The portion of the county west and south from these moraines is a till plain. The morainic ridge on the east border of the county, as well as the interlobate moraine, rises above the 800-foot contour, but the greater part of the plain in the county falls below 700 feet, and on the immediate border of the Wabash scarcely exceeds 600 feet. The valley of the Wabash passes through the county a little south of the center. Occupied as it was by the lake outlet, it is very much larger than the other valleys of the county and the valleys of similar-sized streams in the newer drift area. The average width is nearly 1 mile and the depth 50 to 100 feet.

In Wabash County, as in Huntington, the general elevation of the rock surface is much higher south from the Wabash River than it is north, and as the altitude of the northern portion is fully as great as that of the southern, the drift deposits of the northern portion are very heavy. The drift of the southern portion is about like that of counties to the east and probably falls below an average of 100 feet. The drift in the northern portion probably in places exceeds 300 feet, the thickness at North Manchester, in Eel River Valley, being 274 feet. In the southern portion, however, there are valleys deeply filled with drift that show a rock surface about as low as that of the northern portion of the county. Such a valley was brought to light by the boring at Lafontaine (Ashland post-office), where the drift is found to be 300 feet in thickness.

Wabash County resembles the counties to the east in carrying a somewhat uniform sheet of compact till. The only gravelly districts worthy of note are a narrow belt along the borders of Eel River and terraces in the valley of the Wabash.

Individual wells.—A boring for natural gas at North Manchester penetrates 274 feet of drift, mainly sand. The drift within the limits

[1] *Eighteenth Ann. Rept.* U. S. Geol. Survey, Part IV, p. 498.

of that village, however, varies considerably. Several flowing wells along Eel River valley penetrate till to a depth of 50 to 70 feet, beneath which is a sheet of water with a head sufficient to rise 10 or 12 feet above the surface. The waterworks supply is from flowing wells 100 feet in depth. In the higher portion of the village wells are ordinarily obtained at 30 or 35 feet, without penetrating much till, but in a few instances they are carried to depths of 100 or 115 feet, largely through till.

On the Mississinewa moraine, east and southeast from North Manchester, wells are often in till to a depth of 100 feet or more before obtaining a strong vein of water. One well, 4 miles east of North Manchester (in sec. 12, T. 29, R. 7 E.), penetrated 156 feet of till.

On the plain between Eel River and the Wabash and west from the Mississinewa moraine wells usually penetrate till to a depth of 30 or 40 feet to obtain a strong vein of water. On the north side of Eel River, west from North Manchester, the interlobate moraine usually presents 50 feet or more of compact till at its surface, through which many wells have been sunk.

At Laketon, in the valley of Eel River, below North Manchester, a well 125 feet in depth is mainly through sand. It is thought by the residents of that village that the sheet of sand passes northward beneath the till of the moraine, but there have been few deep wells to test the value of this opinion.

At the city of Wabash wells are often obtained in the valley at depths of 15 to 25 feet without reaching the bottom of the gravel. A better supply is, however, obtained at about 100 feet in the limestone. Some objection is offered to the water from the limestone because of its extreme hardness. For that reason the waterworks are supplied largely from drift wells. The wells are located in the valley of Treaty Creek, south of the city, and are overflowing. The depths range from 42 to 55 feet.

On the Mississinewa moraine, south of the Wabash, several wells have been put down to depths of 95 or 100 feet or more through till before obtaining an adequate supply of water, but as a rule water may be obtained at less than 50 feet.

On the plain west of the Mississinewa moraine, in southern Wabash County, till is usually penetrated to a depth of 25 to 40 feet or more.

At Lafontaine, in the south part of the county, the gas boring penetrates 300 feet of drift, but rock outcrops within a short distance, both above and below Lafontaine, in the bluffs of the Mississinewa River. Wells are usually obtained at Lafontaine and at other points along the Mississinewa, in the southern part of the county, at depths of 25 or 35 feet. Gas borings in the vicinity of Somerset, in the southwest corner of the county, show the drift to have a thickness ranging from 35 up to about 100 feet the lower portion of which is a water-bearing sand or gravel.

n the Seventeenth Report of the Indiana Geological Survey many ords of wells are published which have a depth of only 10 to 40 t. With these are occasional records of deeper wells. One near w Madison (sec. 11, T. 29, R. 7 E.), 207 feet in depth, strikes lime- ne at 181 feet. It penetrates till 160 feet, below which is 21 feet of d. Two other wells in section 26 of the same township penetrate a ilar amount of till, and obtain water in sand or gravel at 164 and feet without entering rock.

t is stated that wells occasionally enter rock at 70 or 75 feet on ms 5 miles north of Lagro, but a well in that vicinity, on the farm Christopher Speicher, strikes no rock at a depth of 156 feet. It is inly through blue till, there being a thin bed of gravel at 80 to 84 t and another at 150 to 156 feet.

Records of two deep wells in the southwest township of the county reported as follows: On farm of Jacob Thomas, depth 64 feet, except 4 feet of water-bearing gravel at bottom; on farm of L. ggoner, depth 96 feet, till 80 feet, below which are alternations of vel, sand, and blue mud, to a limestone at bottom.

On the farm of John H. Pefley, near the south bluff of the Wabash er, in sec. 18, T. 27, R. 8 E., a well 62 feet in depth is reported to e barometric properties of a marked character, there being an ush of air in fair weather and an escape of air in foul weather. A istle, made of two convex disks with a hole in the center, attached gas pipe screwed into the base plate of the pump may be heard quently at the distance of one-half mile, and makes apparent the e of the current of air. It is probable that air spaces in the sand r a medium for the inflow of air to correspond with the changes tmosphere outside the well. The section of the well is as follows:

	Feet.
oil and yellow clay	7
andy blue clay	24
Blue hardpan clay	1½
Dry sand	8
andy blue hardpan	12
Water-bearing blue hardpan	9½
Total	62

MIAMI COUNTY.

General statement.—Miami County is situated in the north-central t of the State, immediately west of Wabash County, with Peru its county seat. Its area is 360 square miles. The broad Wabash lley traverses it nearly centrally from east to west and, as in bash County, separates a district of thin drift on the south from of thick drift on the north. The northern portion is even more vated than the southern, but the rock surface is much lower in the mer *district. The portion* of the county south of the Wabash is

nearly all plane surfaced, and so, also, is much of the district between the Wabash and Eel rivers. That north of Eel River is strongly morainic, and embraces part of the large Erie moraine, which, with the Maxinkuckee moraine of the Saginaw lobe, forms the interlobate moraine of northeastern Indiana and southeastern Michigan. The drift is composed largely of till in the plains, but in the moraines there are knolls of gravel and sand associated with the till knolls, as well as extensive deposits of sand and gravel in the deeper portion of the drift.

Individual wells.—A gas boring 2½ miles north of Peru penetrates 318 feet of drift, a large part of which is water-bearing sand and gravel. Neighboring wells for water usually obtain an abundant supply after penetrating till 30 to 40 feet. One well, however, in sec. 29, T. 28, R. 5 E., penetrated 90 feet of till before obtaining water.

In the city of Peru there are few wells drilled into rock, although there are numerous rock outcrops in the vicinity of the city. The average depth of the wells is about 30 feet, with a range from 15 up to 60 feet. They penetrate either a loose alluvial deposit before entering gravel and sand or pass immediately into gravel and sand. There is, therefore, a liability of contamination of the water from cesspools or other sources. The city water supply is pumped from the Wabash River.

At Bunker Hill a sheet of till 25 to 40 feet in thickness is usually passed through before water-bearing sand or gravel is reached. In one of the gas wells at Bunker Hill the drift was 68 feet, in another 84 feet; in both the lower half was largely sand and gravel.

A gas boring east of Bunker Hill, in sec. 21, T. 26, R. 5 E., at an altitude of 175 feet above the Wabash at Peru, penetrates 60 feet of drift, of which 54 feet is till and the lower 6 feet gravel.

At Amboy the wells penetrate about 15 feet of till, beneath which is a water-bearing gravel extending to the rock. In that vicinity rock is struck at 35 to 50 feet.

At Xenia (Converse post-office) the dug wells are 10 to 20 feet in depth, in gravel beneath a sheet of till. The drift in that vicinity is variable in depth, ranging from 25 up to 100 feet or more in thickness. A few wells are drilled into the rock to depths ranging from 100 to 400 feet. From the deep wells the head is sufficient to bring to the surface water which is reported to be of excellent quality.

In the southwestern part of the county wells are often 50 to 70 feet through till, though shallower wells are usually obtained.

A well at Denver, in the northern part of the county, is noted by Mr. Gorby,[1] which penetrates 125 feet of sand and gravel and reaches no rock.

[1] Sixteenth Ann. Rept. Indiana Geol. Survey, p. 173.

CASS COUNTY.

General statement.—Cass County is situated in the north-central part of the State, the city of Logansport being its county seat. Its area is 420 square miles. The valley of the Wabash traverses it nearly centrally from east to west. The portion south of the Wabash is a comparatively smooth till plain. The portion north of the Wabash is largely morainic, though there is a plain in the extreme northwest corner of the county. There is not the conspicuous descent of the rock surface from south to north that appears in the three counties along the Wabash east from this county. In the immediate vicinity of the Wabash the drift is thin, but on the upland, both north and south of the river, a thickness of 75 to 100 feet or more is found. The drift is very largely till, as in counties to the east.

Individual wells.—At Royal Center, in the northwest part of the county, the oil-well borings enter rock at about 110 feet. The water wells are about 40 feet, though having a range from 15 up to 110 feet. They usually penetrate the following series:

	Feet.
1. Surface clay	4 to 6
2. Gravel	5
3. Blue till, extending to the water-bearing gravel	40
Total	50

At Logansport wells range in depth from 12 to 80 feet, but the majority are about 40 feet. The rock surface being uneven, wells in some cases reach a depth of 40 feet in drift, while in others they enter limestone at 10 or 20 feet. The drift in this city being composed mainly of gravel and the water surface being but a few feet below the well mouth, there is much danger of contamination. The city waterworks obtain the supply in part from springs and in part from Eel River.

On the Wabash bluff, south from Logansport, wells are about 40 feet in depth, the upper half being through till and the lower in gravel.

A well on the north bluff of Wabash River, near Logansport, sec. 27, T. 27, R. 2 E., was largely through sand and gravel to a depth of 90 feet, and struck wood and leaves near the bottom.

In the vicinity of Lake Cicott, in the western part of the county, there are sand deposits on the north bluff of the Wabash which in some cases afford water for wells at a depth of 15 feet. Wells usually, however, penetrate the underlying till.

At Walton, in the southeastern part of the county, the drift is about 80 feet, and is largely till. At Galveston the drift in a valley 20 feet or more below the level of the railway station is shown by a gas boring to be 41 feet in thickness. The lower portion contains much water.

WHITE COUNTY.

General statement.—White County is situated in the northwestern part of the State, immediately west of Cass County, its county seat being Monticello. It has an area of 500 square miles. With the exception of a few drift ridges in its southern portion and occasional sand ridges in its northern portion, it has a plane surface. The drift ridges are most conspicuous immediately east of Chalmers. It is at this point that the large Erie moraine, which has been referred to as occupying the northern portion of Miami and Cass counties, makes a curve to the south. A smaller and perhaps older moraine, formed probably by the Saginaw lobe, leads from near Chalmers westward through White and Benton counties. The Tippecanoe valley in its course across White County deepens from a comparatively shallow ditch near the north border of the county to a trench 75 or 100 feet in depth at the south. Its breadth is but a few rods greater than the stream, and is in strong contrast with the large valley of the Wabash, into which it discharges.

The drift in the northwestern portion of this county, like that of adjacent portions of Pulaski and Jasper counties, is very thin, and the majority of wells enter rock at 10 to 20 feet, or even less. In the southeastern portion of the county there is a heavy deposit of drift, 200 feet or more in thickness.

Individual wells.—At Monon, in the north part of the county, rock is entered at from 3 to 30 feet. Wells are usually about 40 feet, though they range from 20 to 200 feet in depth. The limestone from which the water is obtained contains a sufficient amount of iron pyrites to affect the water appreciably, though not sufficient to render it unfit for drinking.

At Monticello, the county seat, wells are usually obtained at 20 or 30 feet, in gravel below till. Tubular wells are somewhat deeper, the deepest being about 140 feet. The city water supply comes from a large well of moderate depth, but thought to be beyond the reach of surface contamination. The gas-well boring at this place penetrated 205 feet of drift, largely till.

At Reynolds, in the central portion of the county, there are a series of sand ridges, in which water is often obtained at a depth of 15 or 20 feet, or even less. Tubular wells reach a depth of 75 to 120 feet without entering rock. One of the tubular wells, at Mr. Van Voorst's, 56 feet in depth, is reported to have entered a black muck at the base of the sand, about 30 feet from the surface, 6 or 7 feet in thickness, beneath which is a blue clay about 15 feet thick, extending to the water-bearing gravel at bottom. A well opposite the post-office found a similar muck at the bottom of the sand at 20 feet, and entered gravel beneath it at 25 feet. Wells not infrequently encounter muck below the sand in this region. A well in the south part of Reynolds, 120

feet in depth, encountered either a bowlder or a rock ledge, which prevented further boring.

Among the deepest wells in the county is William Owen's, about 6 miles southwest of Reynolds, 190 feet in depth. It entered rock at 30 feet. Inflammable gas was found at about 145 feet. Wells in that vicinity often strike rock at 20 or 30 feet. The thick drift seems to set in immediately east from a line running south from Reynolds through Chalmers. In Chalmers rock is struck at about 90 feet, but at Albert Gosley's, 1½ miles east of Chalmers, no rock was found at 150 feet; and a well at J. N. Bunnell's, a mile southeast of Reynolds, strikes no rock at 160 feet.

On the drift ridge west from Chalmers rock is usually struck at 75 or 80 feet. On the plain north of the ridge it is struck at 20 to 40 feet. On the south slope of the ridge, in sec. 5, T. 25, R. 5 W., rock is struck at 54 feet and water obtained at 75 feet. One well, however, on the moraine near the west line of the county, failed to reach rock at 113 feet. The drift, both on the plain and in the morainic ridges in the southern part of White County, appears to be mainly till.

BENTON COUNTY.

General statement.—Benton County is situated on the western border of the State, immediately south of Newton and Jasper counties and west from White County. It has an area of 503 square miles, and the village of Fowler is its county seat.

This county is occupied by several somewhat prominent drift ridges. In the north tier of townships there is a sharp drift ridge, scarcely a mile in average width, but 25 to 75 feet in height, passing entirely across the county and forming the south border of a plain which is drained by the Iroquois River. A few miles farther south a prominent ridge enters Benton County from the west and passes eastward, bearing slightly north, to the village of Fowler, where it abruptly terminates. This ridge has a breadth of 3 or 4 miles and a relief of 60 to 75 feet or more above bordering plains. On the south border of the county is a third ridge, which passes eastward to the vicinity of Oxford, where it swings abruptly to the north and terminates immediately east of Fowler. This ridge has about the same breadth as the one north of it, but scarcely so great relief. Aside from the three main ridges just mentioned, there are, northwest from Fowler, a series of sharp drift knolls, 25 to 50 feet in height, filling a portion of the plain between the north and middle ridges. Notwithstanding the occurrence of several prominent ridges, the general elevation of the county is remarkably uniform, the variations in elevation being about that of the relief of the ridges. Nearly the entire county lies between the contours of 700 and 800 feet above tide. A point near Fowler, on a sharp gravel knoll, reaches 857 feet, said to be the highest in the county.

The thickness of the drift in this county is found to be very great in the southern and western portions, in one place exceeding 400 feet. But in the northern and eastern portions of the county, including nearly half its area, the drift is seldom found to exceed 100 feet. The surface portion of the drift appears to be generally a compact till, but the deeper portions, so far as tested by well borings, consist mainly of sand and gravel.

Individual wells.—The following list of well sections includes several which have already been published,[1] but the majority were personally obtained from well drillers or owners of wells.

The drift in northwestern Benton County, north from Sugar Creek, is shown by several wells to be a compact till to considerable depth, but along Sugar Creek and on the plain to the south much sand is encountered.

At Earl Park a well, bored by P. M. Crane, penetrates—

	Feet.
1. Soil and yellow till	18
2. Blue till	40
3. Fine gray quicksand	20
4. Blue till	12
5. Limestone	12
Total	102

The water supply is from the limestone. At Raub's warehouse, in Earl Park, limestone was struck at 80 feet, and water obtained at 110 feet. The drift was entirely till. Rock is quarried on Sugar Creek, 1 mile north of Earl Park, on ground but 40 to 50 feet lower.

In sec. 1, T. 26, R. 9 W., near the north line of the county, Mr. Schlautenhofer has a well 108 feet in depth, which obtained water at that depth beneath a bed of dark material thought to be coal. Shale was struck at 70 feet.

In sec. 7, T. 26, R. 8 W., a well 109 feet in depth has the following section:

	Feet.
1. Soil and yellow clay	6
2. Blue clay	44
3. Sand	2
4. Blue clay	57
Total	109

Other wells on the same section are obtained at 35 or 40 feet in a bed of gravel beneath the blue clay.

On Mr. Raub's farm, 2 miles east of Earl Park, water is obtained in limestone at 54 feet. The drift, 50 feet, is entirely till.

On the crest of the north drift ridge, in sec. 20, T. 26, R. 7 W., Mr. J. W. Swan has a well 116 feet in depth, which is thought to have struck sandstone at 82 feet, though it is reported to obtain water from gravel

[1] Fifteenth Ann. Rept. Indiana Geol. Survey, pp. 216-220.

at the bottom. The drift was till to a depth of 75 feet. Another well, only 150 yards distant, penetrated till to a depth of 82 feet without entering rock. On the south slope of the moraine Mr. Swan made a well which obtained water beneath the till at 25 to 27 feet. Another well on the south slope penetrated beds as follows:

	Feet.
1. Till	21
2. Cemented gravel	10
3. Water gravel	3
Total	34

Two miles east from Mr. Swan's, in the western edge of Gilboa Township, a well on the moraine strikes rock at 173 feet. It is reported to be mainly blue till with occasional beds of sand. Water was obtained at a depth of 196 feet. There is a quarry on Pine Creek, just south of this moraine, in sec. 28, T. 26, R. 7 W., at an elevation perhaps 75 feet lower than the well mouth. The rock surface, therefore, varies about 100 feet in elevation.

In the eastern part of the county several wells enter rock at 50 feet or less, and there is a quarry on Pine Creek just north of the village of Aydelott.

A well near the northern terminus of the southernmost of the three ridges above mentioned, in sec. 15, T. 25, R. 7 W., enters rock at 55 feet and obtains water at 60 feet. Another well, in sec. 10 of the same township, 85 feet in depth, enters rock at 72 feet. In both wells the drift is a compact blue till.

At Fowler the waterworks' supply is obtained from wells about 175 feet in depth. They enter rock at 130 feet. The water rises within 25 feet of the surface and the wells are estimated to yield 150 gallons per minute. Several other deep wells have been made in Fowler and vicinity, which show a considerable variation in the elevation of the rock surface. A well at Mr. Van Nata's, in the west edge of the village, enters rock at 91 feet, after penetrating a nearly solid bed of till. A neighboring well, bored by Mr. Fowler, only 300 yards distant, reached a depth of 150 feet without encountering rock. In this well the upper 100 feet is till, below which is gravel and sand. The well at the post-office in Fowler strikes no rock at 160 feet. The drift is reported to be mainly sand. At the Tremont Hotel a well 104 feet in depth strikes no rock and is mainly through sand. The gas-well boring in Fowler, one-fourth mile north of the railway station, strikes rock at 158 feet. In all these wells the surface altitude is very uniform, there being scarcely 10 feet variation. Wells for a short distance north and east of Fowler are obtained at 20 to 30 feet in sand and gravel below till.

On the farm of Mrs. Sumner, at Sugar Grove, 6 miles southwest of Earl Park, rock is struck at 41 feet and water obtained at 47 feet.

A few miles southwest of Fowler, on the middle drift ridge noted

above, several deep wells have been made. Mr. William Bennett, sec. 33, T. 25, R. 9 W., has had several wells sunk to a depth of about 150 feet, and one to a depth of 264 feet, without reaching rock. The drift is mainly till to a depth of 150 feet, below which is sand. The following section is from the well at Mr. Bennett's residence:

	Feet.
1. Soil and yellow till	20
2. Blue till	27
3. Hard gray till	12
4. Greenish-gray till	81
5. Fine gravel	4
6. Greenish till	18
7. Coarse gravel, with water	8
Total	170

A well at Lawrence Broes's, sec. 4, T. 24, R. 9 W., 260 feet in depth encounters no rock, and has a section similar to Mr. Bennett's. A well 3 miles north of Ambia, on the crest of the moraine, penetrates—

	Feet.
1. Yellow till	14
2. Blue till	80
3. Fine, dry sand	20
4. Blue till	46
5. Gravel, with water	1 to 2
Depth to water	160

At John Shilling's, 4 miles northeast of Ambia, near south border of the moraine, a well 114 feet in depth penetrates—

	Feet.
1. Soil and sand	28
2. Blue till	50
3. Quicksand	16
4. Blue till	20
Gravel at bottom.	
Total	114

Wells along the south border of the middle ridge are usually obtained at shallow depths, but occasionally reach a depth of 50 to 65 feet. They often encounter sand and gravel near the surface. Thus, at James Siddon's well, about 2 miles north of Talbot, the drift is mainly sand to a depth of 24 feet, below which there is a bed of blue till 40 feet in thickness, water being obtained in gravel at 65 feet. Another illustration is found in the well at Mr. A. K. Dill's, 3 miles north of Boswell. This well penetrates a loose gravel 18 feet, below which is a thin bed of blue clay, which in turn is underlain by a cemented gravel 40 to 45 feet in thickness, extending to the water gravel at 65 feet.

In the southern tier of townships several borings have been made which show 300 to 400 feet of drift. At Otterbein, however, a boring *212 feet in depth* strikes rock at 132 feet, and about 5 miles north of

Otterbein rock is struck at 80 feet. Among the borings showing a large amount of drift are the following:

W. J. Templeton's boring for artesian water, about three-fourths of a mile northeast of Templeton, reached a depth of 300 feet without entering rock. The drift is mainly till, but sand in its lower part furnishes water, which rises nearly to the surface, 675 feet above tide.

A boring by W. J. and L. Templeton, on their farm in sec. 32, T. 24, R. 8 W., about 5 miles southwest of Oxford, made with a view to obtaining coal, penetrated 372 feet of drift, as follows:

	Feet.
1. Yellow till	12
2. Blue till	115
3. Cemented gravel	25
4. Yellow till and gravel	110
5. Black material, called shale	10
6. Clay and sand	100
Total	372

The well was continued to a depth of 537 feet, or 165 feet into the rock, which is mainly limestone.

A boring for gas on Joseph Atkinson's farm, about 1 mile east of the Templeton well, penetrated 410 feet of drift. A small amount of gas was found in the drift at a depth of 40 feet.

A well in the village of Talbot, 310 feet in depth, strikes no rock; the upper 60 feet is till, the remainder mainly sand.

In addition to the borings showing a very deep drift there are several others worthy of note in this southern tier of townships. A well made by W. J. Templeton, 2 miles east of Templeton, 177 feet in depth, penetrates—

	Feet.
1. Yellow till	12
2. Blue till, with thin sand beds	75
3. Cemented gravel	20
4. Yellow clay and sand	70
Total	177

J. D. Stengle's well at Templeton penetrates 60 feet of till, beneath which is water-bearing sand. Depth of well 80 feet. A well 2 miles south of Boswell penetrates—

	Feet.
1. Yellow till	13
2. Blue till	34
3. Quicksand	20
4. Gravel	66
Total	133

A well on Samuel Dove's farm, immediately east of Boswell, 185 feet n depth, strikes no rock.

At the several villages along the Lake Erie and Western Railroad wells are usually obtained at 20 feet or less, but, as indicated above,

occasional wells are much deeper. Many wells in Templeton obtain water at 20 or 30 feet. They enter blue till at about 6 or 8 feet. At Oxford wells are usually but 14 to 20 feet in depth, though occasionally 80 or 90 feet. In the east part of the village a flowing well has been obtained at a depth of 55 feet. It has a head about 5 feet above the surface. The water is strongly chalybeate. At Ambia, records of two wells were obtained which slightly exceed 100 feet in depth. They are mainly through blue till.

WARREN COUNTY.

General statement.—Warren County is situated on the west border of the State, immediately north of the Wabash River, with Williamsport as its county seat. It has an area of 360 square miles.

Like Benton County, which borders it on the north, this county is traversed by several conspicuous ridges of drift. Along its northern border is the southernmost of the ridges described in the account of Benton County. It has a breadth of 2 or 3 miles and a relief above the plain on the south of 40 to 50 feet or more. A few miles southeast of this ridge is a larger one, which enters the State near State Line village, passes northeastward nearly to Williamsport, and there turns north along the west side of Pine Creek. This ridge is 3 to 6 miles in width and has a relief of 30 to 60 feet or more above bordering plains on the north and south. East from Pine Creek are two ridges trending from northwest to southeast, extending from the valley of the creek to the valley of the Wabash, a distance of 8 or 10 miles. They are each a mile or more in width, and stand 30 or 40 feet above the bordering plains. The narrow plain between them is scarcely 1 mile in average width. The extreme northeastern part of the county is a plain. It is estimated that fully one-half the area of the county is occupied by the drift ridges.

The surface portion of the drift throughout the county is usually a compact till, though the portion east from Pine Creek becomes gravelly in limited areas. Along the Wabash there are conspicuous gravel terraces below Williamsport. Above that city the valley is too narrow for extensive development of terraces. The narrowness is due to the resistant rock strata in which the stream has cut its passage. It is, however, seldom less than one-half mile in width.

For some distance north from the Wabash, unless it be at the extreme eastern border of the county, rock is encountered at comparatively slight depth, seldom more than 100 feet; but along the northern border of the county it is probable that the drift, like that of the adjoining portion of Benton County, is 300 to 400 feet or more in thickness.

Individual wells.—But few well records have been obtained in this county, owing partly to the fact that wells are generally obtained at comparatively slight depth.

At Williamsport, the county seat, wells vary greatly in depth, there

being a range from 14 feet up to about 100 feet. In a portion of the city rock is struck at 10 to 15 feet, but there are places where the drift attains a thickness of 85 feet. A boring for gas at this city 1,540 feet in depth found fresh water at a depth of 65 feet in great quantity; also at about 165 feet. From the latter depth water rises within 32 feet of the surface. Salt water was struck at about 1,200 feet. Gas not being obtained, the salt water was shut off, and the fresh water is thus available for water supply. The well is estimated to yield 20 gallons per minute.

On the Wabash terrace below Williamsport, which stands about 100 to 110 feet above the river, several wells have been sunk to the level of low water in the river. They are through gravel or sand their entire depth. The water rises and falls with the rise and fall of the river.

A boring for coal at West Lebanon penetrated a soil containing wood and leaves beneath the blue till at a depth of 140 feet. Rock was entered a short distance beneath this soil. About a mile southeast from West Lebanon, on ground nearly 100 feet higher, rock is struck at a depth of only 60 feet.

TIPPECANOE COUNTY.

General statement.—Tippecanoe County is situated somewhat north of the middle of the western part of the State, with Lafayette, the county seat, near its geographic center. It has an area of 500 square miles. The greater part of the surface is plain or but gently undulating. There are, however, in the southern half a number of sharp gravelly knolls and ridges, which in places constitute a nearly continuous belt. The most conspicuous one, known as High Gap Ridge, leads from Culver Station westward past Taylor and West Point to the vicinity of the Fountain County line. It is, however, less than a mile in width, and the height of the ridges and knolls seldom exceeds 50 feet. The most conspicuous topographic feature of the county is the troughlike valley traversed by the Wabash River. The valley proper has a depth of more than 100 feet. From its borders there is a gradual rise both to the northwest and southeast—to an altitude in the northwest more than 200 feet above the river, and in the southeast to fully 300 feet.

With the exception of the sharp gravelly knolls referred to above, the surface portion of the drift is composed largely of compact till. The wells of the county usually find water at convenient depths, seldom penetrating so much as 80 feet. The lower portion of the drift, so far as tested by deep borings and by exposures along the Wabash and its tributaries, is largely sand and gravel.

The thickness of the drift varies greatly, partly because of difference in surface elevation, but more largely because of inequalities of *the underlying rock surface.* In the western part of the county, in

the vicinity of the Wabash River, rock ledges rise to a height of 50 to 100 feet or more above the stream, and are well exposed on Flint Creek and Indian Creek. In the northeastern part of the county, also, there are rock exposures on the borders of the Wabash and the lower course of Sugar Creek. Between these exposures, occupying the central portion of the county, there is a deeply filled basin, with drift 300 feet or more in thickness. This appears to be part of a great valley or trough which leads westward across southern Benton County.

To sustain the view that a deeply filled valley extends westward across northern Tippecanoe County, two well sections in the northwestern part of the county are cited. One, in sec. 26, T. 24, R. 6 W., reaches a depth of 176 feet without encountering rock; another, in sec. 18, T. 24, R. 5 W., strikes no rock at a depth of 126 feet. Toward the south, near the Wabash River, the rock appears in valleys of streams at a higher level than the botton of these wells. Passing northward into White County the rock appears at higher level than at these wells. It seems probable that the deepest portion of the valley passes near Dayton and Lafayette and thence north of west across the northwest part of the county, beyond which it finds its continuation westward through southern Benton County.

Along the valley of the Wabash there are gravel terraces reaching a height of nearly 100 feet above the stream. The most conspicuous are located below Lafayette, where they receive the name Wea Plain. They are also conspicuous in the vicinity of Battle Ground village, above Lafayette.

Individual wells.—Wells on the terraces of the Wabash near Battle Ground usually obtain no water within 50 feet of the surface, and in some instances are put down to a depth of 60 or 80 feet. Mr. J. M. Hicks's well, in the village of Battle Ground, is 79 feet, entirely through gravel, sand, or other alluvial deposits. Mr. J. P. Clute's well, in the same village, is 60 feet, striking blue till near the bottom. Wells east from Battle Ground, near the mouth of Moots Creek, are not infrequently 75 feet in depth, and are largely through sand and gravel.

On the uplands west from Battle Ground wells usually are obtained at about 20 or 30 feet, but are occasionally deeper. The following sections of wells in that region have been published by Mr. Gorby:[1]

John Livingston's well, 2 miles west of Battle Ground, penetrates—

	Feet.
1. Soil and yellow clay	30
2. Dry sand	20
3. Blue clay	30
4. Cemented gravel	2
5. Loose gravel	13
Total	95

[1] *Fifteenth Ann. Rept. Indiana Geol. Survey*, pp. 87-95.

James Bryant's well, 3 miles west of Battle Ground, penetrates—

	Feet.
1. Soil and yellow clay	4
2. Blue clay	50
3. Dry sand	20
4. Coarse gravel	2
Total	76

East of the Wabash, in the northern portion of the county, in the vicinity of Buck Creek village, wells are reported by Mr. Gorby to be occasionally 50 to 65 feet in depth. At John Stanfield's a well 65 feet penetrates sand nearly 50 feet before obtaining a coarse water stratum. At Moses Cole's, about 2 miles west of Buck Creek Station, a well penetrates 55 feet of gravel and sand, entering a blue till near the bottom. In Buck Creek village, W. W. C. Brown's well, 50 feet in depth, penetrates till 20 feet, beneath which is a fine yellow sand.

In eastern Tippecanoe County wells are usually obtained at 20 to 40 feet. Mr. Gorby reports an unusually large amount of yellow till in that region, it being frequently 23 to 30 feet in depth.

The gas-well boring at Dayton, in the eastern part of the county, penetrates about 300 feet of drift, of which a large part is thought to be sand.

At Lafayette an artesian well, in the valley of the Wabash, about 55 feet above low-water mark, or 560 feet above tide, penetrates 170 feet of drift and obtains a flow of water from the Lockport limestone at 230 feet. The section of this well published by Mr. Gorby shows variable beds of sand, gravel, and clay to a depth of 30 feet, where a dark-gray clay is entered which extends to the depth of 102 feet. Beneath this there is a series of sand, gravel, and clay beds about 30 feet, below which is a very bowldery bed, 40 feet in thickness, extending to the rock. An analysis of the water from this well appears elsewhere.[1]

The city water supply of Lafayette is obtained from a gravel bed under the Wabash River, about 40 feet below the bed of the stream. The water is found in such abundance at this horizon that the springs which arise from it furnish a considerable portion of the water flowing in the Wabash in dry seasons.

Private wells in Lafayette range in depth from 10 feet up to 60 feet, but are most numerous at 40 to 60 feet. As the city is located on a hillside, with variations of nearly 200 feet in elevation, the wells are obtained from several different horizons. Exposures of the drift are quite extensive in the southern part of the city. They uniformly show a large amount of sand and gravel beneath a comparatively thin sheet of till.

On the gravel terrace in west Lafayette, near Purdue University, which stands about 100 feet above the river, wells are often sunk to the level of the river bed through the sand and gravel of the terrace.

[1] *Eighteenth Ann. Rept. U. S. Geol. Survey*, Part IV, p. 498.

On the portion of the terrace below Lafayette, known as the Wea Plain, wells are frequently put down to a depth of 100 feet or more, thus reaching the level of the river. In the northeastern portion of the Wea Plain wells encounter a blue till before reaching the level of the river, but throughout much of the terrace the gravel extends below river level. A well on this terrace reported by Mr. Gorby has the following section:

	Feet.
1. Soil	3
2. Fine gravel	40
3. Sand and gravel	32
4. Gravel	40
Total	115

This well is located about 1½ miles south of the river in sec. 30, T. 23, R. 4 W. It is stated that at points nearer the river wells in some instances go to a depth of 125 or 130 feet before obtaining water. At points more remote from the river water is obtained at less depth.

In the southern part of the county wells are quite generally obtained at slight depth, seldom exceeding 30 feet and often but 12 to 20 feet. In this region there is very often a bed of water-bearing gravel or sand at the base of the yellow till 10 to 15 feet from the surface, which affords sufficient water for wells. In some cases, however, wells are carried into the underlying blue till and obtain water at depths of 30 to 40 or 50 feet.

CARROLL COUNTY.

General statement.—Carroll County is situated in the north-central portion of the State and includes the part of Wabash Valley immediately above Tippecanoe County. Delphi is its county seat. Its area is 370 square miles. The greater part of the county lies on the east side of the Wabash River. On the west there is a narrow wedge of upland occupying the interval between the Wabash and Tippecanoe rivers. There is a continuous ascent from the east bluff of the Wabash southeastward beyond the limits of the county, an altitude of over 800 feet above tide being attained in its southeastern corner. The immediate bluffs of the Wabash are only 675 feet, while the stream is about 125 feet lower.

The surface of the county is either plane or gently undulating, if we except a few sharp drift knolls in the extreme northwest corner. The tributaries of the Wabash have all shallow channels. There is therefore no conspicuous topographic feature within the county except the Wabash Valley, which has a breadth of a mile or more and a depth of 125 to 150 feet. This valley is bordered by gravel terraces similar to those of Tippecanoe County, the highest of which stand fully 100 feet above the level of the river. Lower terraces occur at several heights.

The thickness of the drift varies greatly, there being quite extensive areas in the vicinity of Delphi, where rock is exposed up to a height

nearly 50 feet above the river. At Monticello, near the northwest ner of the county, the rock floor is found at a level about 100 feet er than the Wabash. Borings in neighboring portions of Howard d Clinton counties on the southeast show drift 200 to 250 feet or re in thickness, the rock floor being lower than at the outcrops ng the Wabash River.

The drift of this county, so far as penetrated in wells, is generally a mpact till. Exposures along the streams also are usually mainly ill. The wells being shallow over the greater part of the county, the ructure of the deeper portion of the drift is not known.

There are very strong springs in the vicinity of Delphi, from which he city water supply is obtained. Their location is about 3 miles northeast of the city, on the farm of George Snyder. The supply is piped to the city and distributed from the standpipe and also by direct pressure.[1]

Individual wells.—Wells are usually obtained at such shallow depths that records of but few of them were obtained. One of the deepest was found at Rockfield, in the northern part of the county, where till to a depth of 57 feet was penetrated and no rock encountered. The ug wells at Rockfield are only 15 to 25 feet in depth. Tubular wells are 50 feet or more.

Along the Wabash terraces, both above and below Delphi, wells are occasionally carried to a depth of 40 feet in gravel and sand; but in he vicinity of Delphi, where rock underlies the terrace at slight lepth, they are sometimes found at the base of the gravel 10 to 20 'eet. On the narrow strip of upland west of the Wabash records of wo wells were obtained, each about 50 feet in depth, one of which vas largely through till and the other largely through sand and gravel, hough only 40 rods apart.

At Flora a gas-well boring penetrates 136 feet of drift thought to e largely till. There are several flowing wells at this village obtaining water at depths of 12 to 47 feet, the distance to the water-bearing tratum differing in the several wells. The wells are located on low round along Bachelor's Run, and their head is only about 5 feet above he surface.

CLINTON COUNTY.

General statement.—Clinton County is situated in the north-central ortion of the State, south of Carroll and east of Tippecanoe County, vith Frankfort as its county seat. It has an area of 400 square miles. This county is an elevated plain, in which the slope is westward and orthward from the southeast part. Its highest points are probably 50 feet above tide, while its lowest are about 700 feet. To the travler, however, the county has the appearance of a nearly level plain, he rate of descent being gradual.

[1] Seventeenth Ann. Rept. Indiana Geol. Survey, p. 172.

The greater elevation of the southeastern portion of the county has been shown by numerous gas-well borings to be due, not to the height of rock surface, but to accumulations of drift, there being about 300 feet of drift in that part of the county.

The surface portion of the drift of this county is a compact till, but gas-well borings show that the deeper portion is largely sand and gravel.

Individual wells.—At Frankfort, the county seat, dug wells are usually obtained at 18 to 25 feet. Tubular wells find a large supply of water at 55 to 65 feet. The wells range in depth from 10 up to 90 feet. In the shallow wells sufficient clay is usually penetrated to insure freedom from surface contamination, though in some cases the water rises so near the surface as to be easily contaminated.

The city water supply of Frankfort is obtained from wells sunk to a depth of about 30 feet below a reservoir which is itself 28 feet in depth. The water fills this reservoir within 3 feet of the surface. Six wells, 8 inches in diameter, are estimated to furnish about 6,000,000 gallons a day. A gas-well boring in the northeast part of Frankfort has the following section:

	Feet.
1. Till	30
2. Quicksand	10 or 15
3. Hard clay and gravel, with flow of water near bottom	25 or 30
4. Gravel and sand, with thin clay beds	197
5. Blue pebbly clay	13
Total	278

A well in the western part of the city did not reach rock at 297 feet. For several miles south and east from Frankfort water is obtained at 15 to 30 feet, being mainly through till, with water-bearing gravel at bottom. North and west from the city the wells are often of greater depth.

At Scircleville a gas-well boring penetrated 296 feet of drift, of which the upper 150 feet is largely till, but the remainder is mainly sand and gravel. Near the east line of the county, about 3 miles east of Scircleville, a gas-well boring enters rock at 260 feet. The drift there is mainly a blue till.

At Kirklin, in the southeast part of the county, a gas-well boring has 252 feet of drift, of which the upper 40 feet is till, but the remainder has alternations of sand and gravel in thin beds, with somewhat thicker sheets of till. The wells in the vicinity of Kirklin obtain a strong supply of water at 40 feet. West from Kirklin wells are often obtained at 16 or 18 feet. Tubular wells are, however, 60 or 70 feet, mainly through till.

At Colfax the deepest wells are 60 or 70 feet, mainly through blue till. Shallower wells are, however, common in that region.

At Mulberry, in the western part of the county, at an elevation 754 *feet above tide,* a water well at the railway station, 220 feet in depth,

id not reach rock. It is mainly through till. Another well at a awmill in the same village is 150 feet in depth, mainly blue till.

Near Gittingsville, in the northern part of the county, several flowng wells have been obtained at about 20 feet. They are along a valey in sec. 19, T. 23, R. 1 E. In section 30 of the same township a vell at R. O. Young's, 71 feet in depth, penetrates till, with thin beds f gravel and sand.

In the vicinity of Sedalia, in the extreme northern portion of the ounty, wells are occasionally sunk to a depth of 80 feet through till, ut are usually shallower. A well at J. H. Brown's, in sec. 24, T. 23, R. 1 W., 126 feet in depth, penetrates—

	Feet.
. Till	68
. Sand, with vegetal deposits and inflammable gas	16
. Sand	42
. Gravel, with water at bottom.	
Total	126

In the Fifteenth Report of the Indiana Survey attention is called to he occurrence of many bowlders in the blue till in Stony Prairie, 5 r 6 miles southwest from Frankfort, from the surface down to a lepth of 50 feet. The western half of the county is liberally strewn vith bowlders, but they seldom extend to such depth as to be troublesome in excavating wells.

HOWARD COUNTY.

General statement.—Howard County is situated in the north-central portion of the State, immediately south of Cass and Miami counties, vith Kokomo as its county seat. Its area is 300 square miles. The entire surface of the county is plain or gently undulating, there being few, if any, knolls and ridges rising more than 20 feet above he general level. With the exception of a narrow belt along Wild Cat Creek, west from Kokomo, which is less than 800 feet above tide, and possibly a narrow strip on the north border, the county stands between the 800 and 900 feet contour.

The greater portion of the county has a moderate coating of drift, 75 feet or less. But the southwestern part has a larger amount, there being at Russiaville and neighboring points about 150 feet. Where he drift is of moderate depth it is composed mainly of till, but the leeper portions include much sand and gravel in their lower part. Wells not infrequently find abundance of water at 10 or 20 feet, in beds of gravel between the yellow and blue till, and they rarely penetrate to a greater depth than 40 feet.

Individual wells.—At Kokomo wells are usually obtained at 35 or 40 feet, either in gravel near the base of the drift or in the upper part of the underlying limestone. A few deep wells 100 to 150 feet have been sunk. The city waterworks are supplied from wells 100 feet in *lepth.*

Several gas borings have been made in the vicinity of Kokomo. In some cases the wells have encountered 90 feet of drift, but there is usually a much smaller amount. Some of those in Wild Cat Creek bottom and on the plain south of the creek enter rock at 10 to 20 feet or less. As a rule, these wells are productive of gas. Those to the west of the city in some cases yield no gas, but, instead, a flow of water. In the Indianapolis Sentinel for July 18, 1895, it is stated that one of these wells had been turned to good account by a farmer for irrigation during a season of excessive drought. Where the water contains only a moderate amount of saline matter such wells might be made to return some small share at least of the expense incurred in drilling for the gas.

At the village of Greentown, 8 miles east of Kokomo, wells are usually obtained at 20 to 25 feet in gravel below till. A lower vein of water is obtained at 35 to 45 feet. Gas-well borings at this point show the drift to be about 85 feet in thickness.

At Fairfield, 6 miles southeast of Kokomo, wells are usually obtained at 20 or 30 feet, there being about 20 feet of surface till. The drift in the vicinity of Fairfield is shown by gas borings to range from 25 feet up to fully 75 feet in depth. The well near the railway station has 47 feet of drift, of which the upper 18 or 20 feet is till and the remainder gravel. A well 4 miles west of Fairfield was found to have 165 feet of drift.

A strong flow of fresh water was obtained from the limestone in a well half a mile north of Fairfield at a depth of 110 feet. It is estimated to discharge 5,000 barrels per day. The head is only 6 feet above the surface of the ground, and the well mouth at this point is 25 feet lower than at Fairfield railway station.

At Tampico, about 3 miles northeast of Fairfield, wells are usually obtained at 30 feet. The drift at that point is shown by a gas boring to be 103 feet in depth, of which the upper 25 feet is till; the remainder consists of alternations of sand and till. A well 3 miles northeast of Tampico, bored for gas, has 115 feet of drift. It is reported to have passed through a bed of soil containing leaves at a depth of 103 to 105 feet.

A gas well at Sylvanus Barrett's, in the southeast township of the county, has 98 feet of drift, of which only the upper 10 feet is till, the remainder being gravel, a part of which is cemented.

At Russiaville, in the southwest township of the county, the gas-well boring penetrates 153 feet of drift, as follows:

	Feet.
1. Yellow till	10 to 12
2. Sand with water	3 to 4
3. Blue till	40
4. Sand and gravel, with thin beds of till	95 to 100
Total	153

Water wells in that vicinity are obtained at shallow depths in the sand between till sheets.

TIPTON COUNTY.

General statement.—Tipton County is situated in north-central Indiana, immediately south of Howard County, with Tipton as its county seat. Its area is 260 square miles. The surface of this county, like that of Howard, is gently undulating, and there is a thicker deposit of drift in its western than in its eastern portion. The western portion is also correspondingly higher. In the southwestern part of the county the altitude exceeds 900 feet, while in the northeastern it is little more than 800 feet. The drift in the southwestern part is 250 to 300 feet in depth, while in the northeastern it is 75 to 100 feet. The surface portion of the drift is generally a compact till, but the deeper portion contains large amounts of sand and gravel.

Individual wells.—At Sharpsville, on the north border of the county, the gas-well borings pass through about 70 feet of drift, mainly till. Water wells are obtained at 20 or 25 feet or less, in thin beds of sand or gravel associated with the till.

At the village of Windfall, in the northeast part of the county, the drift at a gas boring is 72 feet. There are, however, thin beds of sand and gravel associated with the till, which afford water for the wells.

On the east border of the county, in the vicinity of Elwood, the drift ranges from 40 feet up to more than 100 feet in the several gas wells which have been sunk. It is mainly a blue till, but with the till there are beds of sand and gravel furnishing water at moderate depths.

In the vicinity of Hobbs Station, and southeastward to the southeast corner of the county, gas wells usually enter rock at about 70 or 80 feet, and the greater portion of the drift is till.

At Tipton, near the center of the county, the gas wells show 140 to 200 feet of drift, the greatest depth being found on the bank of Cicero Creek, in the east part of the city. The dug wells are often obtained at shallow depths, 12 to 20 feet, and it is thought they are liable to surface contamination, as the drift is of a porous, gravelly constitution. Better wells are found at 50 to 100 feet, after penetrating a sheet of till. Fresh water may be obtained from the limestone, also at several horizons down to about 400 feet, below which depth it is liable to become brackish. The city water supply is derived from near the top of the limestone.

At Kempton, in the western part of the county, water is usually obtained at about 35 feet, in the gravel between sheets of till. Several gas wells have been sunk in the vicinity of Kempton, which show the drift to be 240 to 300 feet in thickness. One of these borings, in the east part of the village, penetrates the following drift series:

	Feet.
1. Yellow till	10
2. Sand and till, each in beds a few feet thick	165
3. Gravel	65
4. Sand	17
Total	257

Another well in the village with 305 feet of drift penetrates—

	Feet.
1. Till, with thin sand beds	200
2. Sand and gravel	80
3. Hard brown till	25
Total	305

A gas well half a mile east of Kempton has 243 feet of drift, consisting mainly of till, but having thin beds of sand and gravel.

At the village of Atlanta a gas-well boring penetrates about 300 feet of drift. Several gas-well borings in the southwest part of the county also penetrate drift 250 to 300 feet.

The following drift section appears in a gas boring near the center of the county, 3 miles east of Tipton:

	Feet.
1. Till, yellow and blue	38
2. Gravel	5
3. Blue till	45
4. Gravel	15
Total	103

A gas well in sec. 10, T. 21, R. 5 E., penetrates 130 feet of drift, as follows:

	Feet.
1. Yellow till	10
2. Gravel	20
3. Blue till	80
4. Gravel	20
Total	130

GRANT COUNTY.

General statement.—Grant County is situated in the north-central part of the State, with Marion as its county seat, and has an area of 420 square miles. It is traversed nearly centrally from southeast to northwest by the Mississinewa River. Along the east border of this stream is the Mississinewa moraine, having a breadth of about 5 miles. Aside from this moraine the surface of the county is a gently undulating plain. The moraine itself has gentle swells and sags and is not strikingly in contrast with the plains. Its relief, however, being 50 feet or more, makes it a noticeable feature.

On the portion of the county west from the Mississinewa River the drift is usually thin, it being often but 20 feet or even less in depth. This district, however, appears to be traversed by deeply filled valleys in which the drift has a thickness of 150 to 160 feet or more. In the northeastern part of the county the drift deposits seldom fall below 100 feet, and probably average nearly 150 feet. Throughout the county the surface portion of the drift is generally a compact till. In places where the drift is thick the deeper portions are often made up largely of sand and gravel.

Individual wells.—At the city of Marion the waterworks are supplied from two classes of wells—first, drift wells 70 feet in depth; second, wells in limestone 240 feet in depth. From both classes of wells the water rises nearly to the surface. A well 8 inches in diameter is estimated to yield nearly 4,000 barrels per day. The drift at Marion ranges from about 30 feet in the north part of the city to 150 feet or more in the south part. The upper portion generally is till to a depth of 30 to 45 feet, the remainder largely sand and gravel.

At Jonesboro, about 6 miles above Marion, on the Mississinewa River, wells are obtained in gravel below till at depths of 10 to 45 feet. The gas-well boring in the village shows the drift to be 162 feet.

At Upland, in the eastern part of the county, wells are usually obtained at about 30 feet, in a bed of gravel between sheets of till. A few wells are sunk into the underlying rock and obtain water at about 150 feet. The waterworks supply is from these rock wells.

At Van Buren the gas well penetrates 92 feet of drift, largely till. Wells in the vicinity of that village are obtained at 20 or 30 feet.

A gas boring in the northwestern part of the county penetrates 182 feet of drift, which is reported to be mainly sand and gravel.

In the southwestern portion of the county, although the drift is thin, wells are usually obtained without entering the rock, there being generally a water-bearing gravel at the base of the drift. Gas-well borings in this part of the county show the drift to have the following thicknesses at the principal villages: At Switzer, 28 feet; at Swayzee, 22 feet; at Simms, 45 feet; at Point Isabel, 22 feet; at Fairmount, 5 to 35 feet, the thickness varying in different wells. At Fairmount water wells are often drilled into the rock to a depth of about 30 feet. The waterworks supply is from such wells.

BLACKFORD COUNTY.

General statement.—Blackford County is situated somewhat north of the middle of the eastern portion of the State, and is one of the smallest counties, having an area of 170 square miles. Its county seat is Hartford City. The Mississinewa moraine traverses the southern townships of the county from west to east, and occupies about 75 square miles. The remainder of the county is a plain. With the exception of a narrow belt in the northeastern part, along the Salamonie River, the drift of this county is about like that of northeastern Grant County—100 to 150 feet or more in thickness. In the northeast corner the thickness is but 20 or 30 feet, or even less. The drift is composed mainly of a compact till, though beds of sand and gravel are sufficiently numerous to supply water for most of the wells at moderate depths.

Individual wells.—At Montpelier, in the northeastern part of the county, wells are in some cases drilled into the limestone, though usually obtaining water at the base of the drift at depths of 15 to 30 feet.

At Mill Grove, in the eastern part of the county, a gas-well boring penetrates 143 feet of drift, mainly till. Water wells are usually obtained at moderate depths, seldom more than 40 feet. In a few instances, however, they reach a depth of fully 100 feet in that part of the county.

At Hartford City the several gas borings show the drift to range in depth from 80 feet up to 150 feet or more. It is composed of till, interrupted by occasional thin beds of sand. The waterworks supply is from wells drilled into the underlying limestone to a depth of about 250 feet. A well at one of the hotels in Hartford City enters rock at 112 feet. The upper 80 feet is a nearly solid bed of till, below which is 32 feet of sand and gravel. A well 1 mile northwest of Hartford City, in section 10, has 177 feet of drift, of which the upper 100 feet is till and the remainder largely sand. One well in the southeast part of Hartford City enters rock at about 80 feet after penetrating 60 feet of till and about 20 feet of sand.

JAY COUNTY.

General statement.—Jay County is situated on the eastern border of the State, somewhat north of the middle, with the county seat at Portland. Its area is 396 square miles. This county is traversed from west to east by two morainic belts, the Mississinewa and the Salamonie. The Mississinewa moraine is confined mainly to the southern tier of townships and has a breadth of about 5 miles. The Salamonie moraine is situated mainly in the northern tier of townships, but enters the northern portion of the middle tier near the State line. Its breadth is 3 to 5 miles. The remainder of the county has a nearly plane surface, sloping gently toward the north. With the exception of a few sharp knolls on the Salamonie moraine near Camden (Pennville post-office), in the northwest part of the county, the moraines have gentle swells and sags, which give them but little contrast with the gently undulating plains. They present, however, a nearly continuous ridge with relief of 30 to 50 feet or more above the bordering plains. The altitude of the southern portion of the county exceeds 1,000 feet. The northern border of the county falls below 900 feet.

The drift is composed mainly of a sheet of compact till, and its thickness probably averages about 100 feet. The rock is rarely struck at less than 50 feet, and at Geneva, near the north border of the county, a gas-well boring shows 350 feet of drift.

Individual wells.—Along the north border of the county, near the State line, rock is found at 10 to 20 feet, and wells are often sunk into it to slight depths. The thin drift, however, occupies but a few square miles. At Bryant, near the middle of the north line of the county, the drift was found to be 112 feet in thickness.

About 6 miles west of Bryant, in secs. 19 and 30, T. 24, R. 13 E., wells are difficult to obtain, the moraine at that point being composed

of till to a depth of 125 feet or more. One well has a depth of 125 feet, another 140 feet, and another 155 feet, none of which enter rock. About a mile west of these wells rock is struck at 130 feet in a well in section 13, Penn Township.

At Pennville, in southern Penn Township, on the Salamonie River, rock is struck at about 40 feet in several gas borings.

At Portland, the county seat, wells are often obtained at 14 to 30 feet in gravel, but more frequently they are drilled into limestone, which is entered at 30 to 60 feet in the vicinity of this town. A few wells have a depth of 150 feet. The waterworks supply is from wells drilled into the limestone. It is thought that the shallow wells are protected from contamination by a bed of clay which overlies the water-bearing gravel.

Immediately east of Portland wells are obtained in sand between yellow and blue tills at 12 to 20 feet; the blue till is struck at 15 to 30 feet, and extends to the underlying rock at 45 to 50 feet.

A well in section 18, Noble Township, 5 miles east of Portland, enters rock at 83 feet; one in section 17 enters rock at 160 feet, and one in section 7 enters rock at 57 feet. They are all situated near the south border of the Salamonie moraine and are mainly through till.

Near Bellefontaine, in the east part of the county, wells enter rock at 80 to 135 feet and are mainly through till.

At Como, 6 miles southwest from Portland, the drift is 80 feet, and at Red Key 73 feet, mainly till at both places.

At Dunkirk, in the southwestern part of the county, the best wells are obtained near the top of the limestone at depths of 65 to 100 feet. The waterworks are supplied from wells of this class. Shallower wells afford only a small amount of water.

Along the Mississinewa moraine, in the southern part of the county, farmers not infrequently have sunk wells to a depth of 50 to 75 feet or more before obtaining an adequate supply of water. This moraine appears to afford but little water at shallow depths.[1]

---

[1] The size of a single paper in this series being limited by law to 100 pages, the remainder of this paper—the portion relating to the southern part of the State—will be published as Water-Supply Paper No. 26 (in press).

INDEX.

Page.

s County, description of wells in... 51-52
orth, description of wells near..... 14
. description of wells at..... 32,34
County, description of wells in.... 48-51
. description of wells near 64,66
y, description of wells at..... 58
ws, description of wells near 54-55
a, description of wells near..... 28-29
ta, description of wells at..... 76
d, description of wells at 36
rn, description of wells near 30
, description of wells near..... 33
Ground, description of wells near. 68-69
ontaine, description of wells near.. 79
n County, description of wells in.. 61-66
ford County, description of wells in 77-78
description of well at..... 16
on, description of wells at..... 53
Grove, description of wells at..... 18
ll, description of well near..... 64,65
en, description of wells near..... 38
leld, description of wells near..... 34
e, description of wells near..... 43
t, description of wells near..... 78-79
Creek, description of wells near.... 69
r Hill, description of wells near... 58
tt, description of wells near..... 37
, description of wells near..... 30
ll County, description of wells in.. 70-71
County, description of wells in..... 59
ers, description of wells near..... 61
berlin, T. C., cited..... 11
erton, description of wells near.... 16
busco, description of wells at..... 48
ool, description of wells near 37
n County, description of wells in.. 71-73
, description of wells at..... 72
s, description of wells at..... 48
bia City, description of wells at... 48
a Point, description of wells near.. 14
n, description of well at..... 69
ur, description of wells at..... 51
b County, description of wells in .. 29-31
i, description of wells near..... 71
r, description of well at..... 58
, C. R., cited 25,26,27,28
irk, description of wells at.. 79
'ark, description of wells near..... 62
rt, description of well at..... 24
rt County, description of wells in.. 23-25
d, description of wells at..... 75
e lobe, description of..... 11
reen, description of wells at 36

Page.

Fairfield, description of wells near..... 74
Fairmount, description of wells at..... 77
Flora, description of wells near..... 71
Fort Wayne, description of wells at..... 49
Fowler, description of wells near..... 63
Frankfort, description of wells near..... 72
Fulton County, description of wells in ... 46-47
Furnessville, description of well at..... 16
Galveston, description of wells at 59
Garrett, description of wells at 31
Geneva, description of wells at..... 52
Gibson, description of wells at..... 14
Gittingsville, description of wells near .. 73
Glacial conditions of Indiana..... 9-13
Goshen, description of wells at..... 24
Grant County, description of wells in.... 76-77
Greentown, description of wells at..... 74
Hageman, description of wells at 16
Hammond, description of wells at..... 14
Hartford City, description of wells near. 78
Hawville, description of wells near 34
Hebron, description of wells at 18
Hessville, description of wells at..... 14
Hobart, description of wells near 14
Hobbs Station, description of wells at ... 75
Howard County, description of wells in.. 73-74
Hudson, description of wells near..... 29
Huntington, description of wells at..... 54
Huntington County, description of wells in..... 54-55
Illinoian drift sheet, description of..... 10
Inwood, description of wells at..... 38
Iowan drift sheet, description of..... 10
Jackson Center, description of wells near. 18
Jasper County, description of wells in... 44-45
Jay County, description of wells in..... 78-79
Jonesboro, description of wells at..... .. 77
Kempton, description of wells near..... 75-76
Kendallville, description of wells near... 32-33
Kentland, description of wells near... 41-42,43
Kewanna, description of wells at..... 47
Kirklin, description of wells near..... 72
Knox, description of wells at..... 39-40
Kokomo, description of wells near..... 73-74
Kosciusko, description of wells at..... 36
Kosciusko County, description of wells in. 35-37
Kouts Station, description of wells at.... 18
Lacrosse, description of wells near..... 21
Lafayette, description of wells near..... 69-70
Lafontaine, description of wells at..... 56
Lagrange, description of wells near 25-26
Lagrange County, description of wells in. 25-27
Lagro, description of wells near 57

Page.
Lake, description of wells at 14
Lake County, description of wells in 13-14
Lake Maxinkuckee, description of wells near 29-30
Laketon, description of wells at 56
Laporte County, description of wells in 18-21
Ligonier, description of wells at 35
Liverpool, description of wells at 14
Loess defined 10
Logansport, description of wells near 50
Marion, description of wells at 77
Marshall County, description of wells in 37-39
Maxinkuckee moraine, description of 11
Medaryville, description of wells at 46
Mentone, description of wells near 37
Metz, description of wells near 29
Miami County, description of wells in 57-58
Michigan City, description of wells near 19-20
Milford, description of wells at 36
Mill Grove, description of wells at 78
Miller Station, description of well near 14
Mishawaka, description of wells at 22
Monon, description of wells at 60
Monticello, description of wells at 60
Montpelier, description of wells at 77
Morocco, description of wells at 43
Mount Ayer, description of wells at 43
Mulberry, description of wells at 72-73
New Haven, description of wells near 50
New Lancaster, description of wells near 54
New Madison, description of wells near 57
New Paris, description of wells near 24-25
Newell, F. H., letter of transmittal by 7
Newton County, description of wells in 40-43
Noble County, description of wells in 31-35
Noblesville, description of wells near 34
North Galveston, description of wells near 38
North Judson, description of wells at 40
North Liberty, description of wells near 23
North Manchester, description of wells near 55-56
Oswego, description of wells near 36
Otterbein, description of wells near 64-65
Oxford, description of wells near 65-66
Pennville, description of wells at 79
Peru, description of wells near 58
Pierceton, description of wells near 36
Pleasant Mills, description of wells at 52
Plymouth, description of wells near 38
Porter County, description of wells in 15-18
Portland, description of wells near 79
Pulaski County, description of wells in 46
Remington, description of wells at 45
Rensselaer, description of wells near 45
Reynolds, description of wells near 60-61
Roanoke, description of wells near 54
Rochester, description of wells at 47
Rockfield, description of wells at 71
Rome City, description of wells near 34
Royal Center, description of wells at 59
Russiaville, description of wells near 74
Saginaw ice lobe, description of 11
St. John, description of wells near 14
St. Joseph, description of wells near 31
St. Joseph County, description of wells in 21-23
Scircleville, description of wells near 72
Sedalia, description of wells near 73
Sharpsville, description of wells at 75
Somerset, description of wells near 56
South Bend, description of wells near 22,23
South Milford, description of well near 37
Spencerville, description of wells near 31,50
Starke County, description of wells in 39-40
Steuben County, description of wells in 27-30
Sugar Grove, description of well at 63
Surrey, description of wells near 44
Talbot, description of well near 64,65
Tampico, description of wells at 74
Teegarden, description of wells near 38
Templeton, description of wells near 65,66
Till defined 12
Tippecanoe County, description of wells in 67-70
Tipton, description of wells at 75,76
Tipton County, description of wells in 75-76
Tolleston, description of wells at 14
Uniondale, description of wells near 53
Upland, description of wells at 77
Valentine, description of well near 26
Valparaiso, description of wells near 17-18
Valparaiso moraine, description of 11
Van Buren, description of wells at 77
Wabash, description of wells at 56
Wabash County, description of wells in 55-57
Walkerton, description of wells near 23
Walton, description of wells at 59
Warren, description of wells near 55
Warren County, description of wells in 66-67
Warsaw, description of wells near 36-37
Waterloo, description of well at 30
Wells County, description of wells in 52-53
West Lebanon, description of wells near 67
Wheeler, description of wells near 17
White County, description of wells in 60-61
Whiting, description of wells at 14
Whitley County, description of wells in 47-48
Williamsport, description of wells near 66-67
Winamac, description of wells at 46
Windfall, description of wells at 75
Wisconsin drift sheet, description of 10
Wolcottville, description of well near 27
Xenia, description of wells at 58

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 22

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

Page.
Lake, description of wells at 14
Lake County, description of wells in..... 13-14
Lake Maxinkuckee, description of wells near 38-39
Laketon, description of wells at..... 56
Laporte County, description of wells in..... 18-21
Ligonier, description of wells at 35
Liverpool, description of wells at..... 14
Loess defined 10
Logansport, description of wells near.... 59
Marion, description of wells at..... 77
Marshall County, description of wells in. 37-39
Maxinkuckee moraine, description of.... 11
Medaryville, description of wells at..... 46
Mentone, description of wells near..... 37
Metz, description of wells near..... 29
Miami County, description of wells in... 57-58
Michigan City, description of wells near. 19-20
Milford, description of wells at..... 36
Mill Grove, description of wells at..... 78
Miller Station, description of well near.. 14
Mishawaka, description of wells at..... 22
Monon, description of wells at 60
Monticello, description of wells at..... 60
Montpelier, description of wells at..... 77
Morocco, description of wells at..... 43
Mount Ayer, description of wells at..... 43
Mulberry, description of wells at 72-73
New Haven, description of wells near.... 50
New Lancaster, description of wells near 54
New Madison, description of wells near.. 57
New Paris, description of wells near..... 24-25
Newell, F. H., letter of transmittal by... 7
Newton County, description of wells in.. 40-43
Noble County, description of wells in.... 31-35
Noblesville, description of wells near.... 34
North Galveston, description of wells near 36
North Judson, description of wells at.... 40
North Liberty, description of wells near. 23
North Manchester, description of wells near 55-56
Oswego, description of wells near..... 36
Otterbein, description of wells near..... 64-65
Oxford, description of wells near..... 65-66
Pennville, description of wells at..... 79
Peru, description of wells near..... 58
Pierceton, description of wells near..... 36
Pleasant Mills, description of wells at.... 52
Plymouth, description of wells near..... 38
Porter County, description of wells in.... 15-18
Portland, description of wells near..... 79
Pulaski County, description of wells in.. 46
Remington, description of wells at..... 45
Rensselaer, description of wells near 45
Reynolds, description of wells near..... 60-61

Page.
Roanoke, description of wells near..... 54
Rochester, description of wells at..... 47
Rockfield, description of wells at..... 71
Rome City, description of wells near..... 34
Royal Center, description of wells at..... 59
Russiaville, description of wells near..... 74
Saginaw ice lobe, description of..... 11
St. John, description of wells near..... 14
St. Joseph, description of wells near..... 31
St. Joseph County, description of wells in. 21-23
Scircleville, description of wells near.... 72
Sedalia, description of wells near..... 73
Sharpsville, description of wells at..... 75
Somerset, description of wells near..... 56
South Bend, description of wells near... 22,23
South Milford, description of well near.. 27
Spencerville, description of wells near... 31,50
Starke County, description of wells in... 39-40
Steuben County, description of wells in.. 27-29
Sugar Grove, description of well at..... 63
Surrey, description of wells near..... 44
Talbot, description of well near..... 64,65
Tampico, description of wells at..... 74
Teegarden, description of wells near..... 38
Templeton, description of wells near.... 65,66
Till defined 12
Tippecanoe County, description of wells in..... 67-70
Tipton, description of wells at..... 75,76
Tipton County, description of wells in... 75-76
Tolleston, description of wells at..... 14
Uniondale, description of wells near..... 53
Upland, description of wells at..... 77
Valentine, description of well near 26
Valparaiso, description of wells near.... 17-18
Valparaiso moraine, description of..... 11
Van Buren, description of wells at..... 77
Wabash, description of wells at..... 56
Wabash County, description of wells in.. 55-57
Walkerton, description of wells near.... 23
Walton, description of wells at..... 59
Warren, description of wells near..... 55
Warren County, description of wells in.. 66-67
Warsaw, description of wells near..... 36-37
Waterloo, description of well at..... 30
Wells County, description of wells in.... 52-53
West Lebanon, description of wells near. 67
Wheeler, description of wells near 17
White County, description of wells in.... 60-61
Whiting, description of wells at..... 14
Whitley County, description of wells in.. 47-48
Williamsport, description of wells near.. 66-67
Winamac, description of wells at..... 46
Windfall, description of wells at..... 75
Wisconsin drift sheet, description of..... 10
Wolcottville, description of well near.... 27
Xenia, description of wells at..... 58

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 22

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

EWAGE IRRIGATION

PART II

BY

GEORGE W. RAFTER

WASHINGTON

GOVERNMENT PRINTING OFFICE

1899

CONTENTS.

	Page.
ır of transmittal	9
mé of Water-Supply and Irrigation Paper No. 3	11
ıtic and geographic distribution of sewage-disposal works	11
n of sewage irrigation	13
ıarison of English and American sewage	13
tion of rivers	15
ge purification at manufacturing establishments	22
achusetts experiments on the purification of factory wastes	27
'aper-mill wastes	27
Vool-scouring wastes	27
'annery wastes	28
)yestuffs	29
pecial experiments	30
Iechanical analyses of sands	31
Size of grain	31
Air and water capacities of sand	33
Limitation of size of dose	33
'urification obtained by intermittent filtration	34
ummary of Massachusetts experiments	34
ı of commercial fertilizers	35
ısity of purification of sewage at towns on the Great Lakes	36
:ffect of sewage on lake waters	37
)anger of winter typhoid	38
ican sewage-disposal plants	41
Iaine	41
Augusta, State Insane Asylum	41
'ew Hampshire	41
Concord, State Insane Asylum	41
Rockingham County Poor Farm	42
Iassachusetts	42
Amherst	42
Brockton	42
Gardner	45
Greenfield	46
Lenox	46
Marlboro	46
Concord, Massachusetts Reformatory	48
Medfield	48
Mystic Valley	48
Natick	50
Norfolk County Jail	51
North Brookfield	51
Pegan Brook	52
Waltham, School for the Feeble-minded	52
Sherborn Prison	52

American sewage-disposal plants—Continued.
Massachusetts—Continued. Pa
South Framingham
Wellesley College
Westboro
Worcester
Worcester Insane Hospital
Plants projected in Massachusetts
Rhode Island
Central Falls
Pawtucket
Cranston, State institutions
Providence
Woonsocket
Connecticut
Bristol
Danbury
Lake Waumemaug
Litchfield
Manchester
Meriden
Waterbury
New York
Brewsters
Brooklyn, twenty-sixth ward
Chautauqua Assembly
Far Rockaway
New Rochelle
Hemlock Lake
Bath, Soldiers' and Sailors' Home
Batavia, State Institution for the Blind
Vassar College
Plants projected in New York
New Jersey
Atlantic City
East Orange
Essex Fells
Freehold
Lawrenceville School
Long Branch
Morris Plains Asylum
Plainfield
Princeton College
Summit
Westfield
Pennsylvania
Altoona
Norristown, State Hospital for the Insane
Reading
Wayne
West Virginia
Weston, State Insane Hospital
Texas
Paris
San Antonio
Temple

xnerican sewage-disposal plants—Continued. Page.
Ohio 75
Alliance 75
Canton 75
Fostoria 76
Oberlin 76
Michigan 77
Wayne County poor farm 77
St. Johns 77
Illinois 77
Pullman 77
World's Fair 79
Wisconsin 79
Menominee Valley 79
Minnesota 79
Rochester Hospital for the Insane 79
Nebraska 80
Hastings 80
Colorado 80
Colorado Springs 80
Leadville 80
Trinidad 81
Montana 81
Helena 81
Utah 81
Salt Lake City 81
Arizona 82
Phœnix 82
California 82
Fresno 82
Los Angeles 82
Pasadena 84
Redding 85
San Luis Obispo 85
Santa Rosa 85
Stockton 85
Dominion of Canada 85
Berlin 85
Hamilton 88
London 88
London Hospital 88
Ontario Agricultural College 89
Waterloo 89
ublications relating to sewage utilization and disposal 89
ndex 99

ILLUSTRATIONS.

Page

PLATE I. Brockton, Massachusetts, filtration fields; *A*, Turnips and sweet corn; *B*, Pease, sweet corn, and beans 42

II. South Framingham, Massachusetts, sewage farm; *A*, Sewage inlet to filter; *B*, Portion of cabbage field 52

III. Works at Worcester, Massachusetts; *A*, General view of precipitation tanks; *B*, Another view of precipitation tanks 54

IV. Filter areas as ridged for winter service at Pawtucket, Rhode Island. 58

V. Sewage disposal at Plainfield, New Jersey; *A*, Main carrier as seen from screen tank (before erection of building); *B*, Distributing channel in bed No. 1 70

VI. Sewage irrigation at Salt Lake City, Utah; *A*, Main ditch, outlet gate, and field of corn; *B*, Lateral ditch and method of irrigating corn. 80

VII. *A*, Sewage irrigation at Phœnix, Arizona; *B*, Sidehill irrigation at Wayne, Pennsylvania 82

FIG. 1.—Mechanical composition of materials available for sewage filtration .. 32

2.—Air and water capacities of sewage filter materials 32

3.—Limitation of size of single dose by air and water capacities 33

4.—Plan of filter areas at Pawtucket, Rhode Island 58

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, October 25, 1898.

SIR: I have the honor to transmit herewith a manuscript entitled Sewage Irrigation, Part II, prepared in October, 1897, by Mr. George W. Rafter, of Rochester, New York. This paper consists in part of material furnished for Water-Supply and Irrigation Paper No. 3, on Sewage Irrigation. In the printing of that paper it was found that the limit of 100 pages set by law would be exceeded in spite of considerable condensation in the form of statement. The original manuscript was therefore divided into two parts, the first, relating to the general subject and giving the practice abroad, having already appeared as Paper No. 3. The present manuscript, which has been considerably extended by Mr. Rafter, is devoted mainly to the discussion of data obtained in this country and in Canada, and of all the plants erected and operated, together with brief notes on the projected plants. There is included an appendix containing a list of publications relating to the subject.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

SEWAGE IRRIGATION, PART II.

By GEORGE W. RAFTER.

RÉSUMÉ OF WATER-SUPPLY AND IRRIGATION PAPER NO. 3.

In a previous publication on sewage irrigation[1] we discussed the general principles governing the utilization of sewage in agriculture, as well as the relations of such utilization to the public health. It was shown, also, how the gradually increasing pollution of streams first led in England and other manufacturing countries, to a demand for sewage purification, and the probable value of sewage in agriculture was indicated.

In that paper the various methods of sewage disposal in common use were discussed, with special reference to the modern views about nitrification. Descriptions of methods of applying sewage were given with information as to the best crops for sewage farming, the proper quantity of sewage to be applied, limiting temperatures, cost of labor on sewage farms, sanitary condition of such farms, and a number of other important subjects which it is necessary that one should understand before embarking in sewage-utilization investments. Brief descriptions were also given of a number of the principal sewage farms in England, of the sewage farms of the city of Berlin, Germany, and of sewage utilization in France on the Plain of Gennevilliers.

In the present paper it is proposed to give additional data relating to stream pollution and sewage purification, together with a brief account, so far as it has been possible to obtain the information, of every sewage-disposal plant in the United States and the Dominion of Canada, whether now in use or abandoned, that has actually been built, or that has had plans prepared for its construction.

CLIMATIC AND GEOGRAPHIC DISTRIBUTION OF SEWAGE-DISPOSAL WORKS.

As regards rainfall, the United States divides naturally into three regions—the humid, subhumid, and arid. The humid region includes all of the country east of the Mississippi River, together with the States of Iowa, Missouri, Arkansas, and Louisiana west of that river.

[1] Water-Supply and Irrigation Paper No. 3, 1897, 100 pp., 4 pls.

It also includes portions of North Dakota, South Dakota, Nebraska, Kansas, Indian Territory, and Texas. The subhumid region may be taken to include the remainder of North Dakota, South Dakota, Nebraska, Kansas, Indian Territory, and Texas. The arid region embraces Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, Utah, Washington, and Wyoming, although limited areas in several of these States and Territories may be classed as humid.

The great lack of water in the subhumid and arid regions would naturally indicate a relatively large development of sewage irrigation in those portions of the United States. As a matter of fact, however, other conditions than mere shortage of water have controlled—for instance, the degree of advancement of the towns themselves—and we accordingly find, on the whole, more sewage irrigation and general purification in the humid East than in the subhumid and arid West. The Western towns are, nevertheless, advancing rapidly, and we may expect changes in the figures in the near future.

The following paragraph gives the approximate statistics of sewage purification in the United States to-day:

In the humid region there are sewage-purification plants in operation, actually building, projected, or built and abandoned, distributed by States as follows: Maine, 1; New Hampshire, 2; Massachusetts, 32; Rhode Island, 5; Connecticut, 7; New York, 33; New Jersey, 11; Pennsylvania, 5; Maryland, 1; West Virginia, 1; Louisiana, 1; Texas, 3; Ohio, 11; Michigan, 2; Illinois, 2; Wisconsin, 2;[1] Minnesota, 1; a total of 120 for the humid region. In the subhumid there is 1 in Nebraska. In the arid region Arizona has 1; Colorado, 4; California, 8; Montana, 1; Utah, 1; Wyoming, 1, or a total of 16. There are also six plants in the humid portion of the Dominion of Canada.[2]

The foregoing figures indicate a total for the United States and Canada of 143. About 120 of these plants have been either built or projected in the last ten or twelve years. This total includes, so far as can be learned, all the purification plants of every kind, whether irrigation, intermittent filtration, or combined chemical-purification and filtration and irrigation plants, either actually in operation now or formally projected and in such a state of advancement as probably to be carried out in a few years.

The intermittent-filtration plants are properly included in a paper on sewage irrigation, because the filtration areas either now are used for raising crops or are likely in the end to be so used. As to the propriety of including the chemical-purification plants, it may be pointed out that this system of purification has been adopted in several towns before all the controlling conditions were taken into account. In one case an

[1] Aside from three plants for purifying manufacturing wastes, etc., there are only two sewage-disposal works in Wisconsin.

[2] *The foregoing statistics* of the distribution of sewage-disposal works were prepared in September, *1897. At the present date* (February, 1899) there are a few additions.

eastern town has adopted chemical purification and operated the plant at large expense, in spite of the existence of ideal conditions for irrigation and filtration in the immediate vicinity. It is probable, therefore, that, as soon as the advantages of irrigation and filtration are more fully known, a number of the chemical-purification plants will be changed to either irrigation or filtration, or, without a complete change of plant, the effluents from chemical-purification works will be further treated by either irrigation or filtration. This proposition is especially true of a number of chemical plants in the State of New York, where the capitalized cost of chemical treatment will be considerably in excess of that of intermittent filtration, even though sand of the proper quality were transported some distance and high-grade filtration areas constructed. It is in this view of the case that the statistics of chemical-purification plants are herein included. There are also a number of cities and villages in the Eastern part of the United States where crude sewage is discharged into ponds, lakes, and running streams, which either now are or are likely in the future to be the source of public water supplies. In the end the sewage of all such will require purification. In many cases proper land areas can be had in the immediate vicinity; hence we find an additional reason why a complete exhibit of the whole subject should be made at this time. All town authorities need to understand that, with other conditions equal, the capitalized cost of the land-purification processes is ordinarily less than that of the chemical. Farmers in the vicinities of towns need also to understand this, as well as the benefits to themselves to be derived from the utilization of sewage in agriculture.

ORIGIN OF SEWAGE IRRIGATION.

The growth in England of great manufacturing industries and the consequent pollution of the streams of that country at the beginning of the present century forced the English cities to consider the necessity for sewage purification at an earlier date than elsewhere. Sewage irrigation as a branch of applied science may therefore be said to have originated in England. At the present time a more general use of sewage purification may be found in that country than elsewhere, although, as we have seen in Water-Supply and Irrigation Paper No. 3, the Germans and the French have now extensive sewage-purification plants of a high order of excellence. Nevertheless it is true that a large proportion of the data of sewage disposal, as it exists to-day, must necessarily be drawn from English practice.

COMPARISON OF ENGLISH AND AMERICAN SEWAGE.

First of all, it is necessary that we establish in some way a measure of the relation which average English sewage bears to American. The great difficulty in making comparisons is the fact that the English

chemists, following the lead of Dr. Frankland, have mostly used the combustion process in their sewage analyses, while in the United States the Wanklyn albuminoid-ammonia process has been almost exclusively used.

By way of explaining the terms used, it may be pointed out that free ammonia as determined by the Wanklyn process represents that portion of the nitrogenous matter present which has already, at the time of analysis, passed into a state of decay; whereas the albuminoid ammonia represents the nitrogenous matter which has not, at the time of analysis, undergone decay. These are the two main determinations of the Wanklyn process.

In the combustion process Dr. Frankland attained much greater refinement than is possible with the albuminoid-ammonia process of Wanklyn, and the only reason why the combustion process has not been universally adopted is because of the difficulty of working it. By its use we determine not only the different states of the nitrogen, but that portion of the organic matter present which is carbonaceous in its character, or, technically, the organic carbon. The organic nitrogen of the process is that portion of the nitrogenous matter due to living substances, and corresponds in a general way with the albuminoid ammonia of Wanklyn. The ammonia represents dead and usually animal matter, and is, in the opinion of Dr. Frankland, the chief index of recent sewage contamination.

There are four states of the nitrogen as found in contaminated water—ammonia, free ammonia, nitrite, and nitrate. The nitrates represent the final fixation of the ammonia by combination with a mineral base, and hence a state of innocuousness. By a comparison of these various states Dr. Frankland arrives at the previous sewage contamination, which is one of the most interesting determinations of his system.[1]

If we refer to the various reports of the Rivers Pollution Commission, the Royal Commission on Metropolitan Sewage Discharge, or the Royal Commission on Metropolitan Water Supply, or to the various reports of the English local government boards, we find the analyses usually given in terms of the combustion process, in which organic carbon, organic nitrogen, ammonia, and total combined nitrogen may be considered the chief controlling constituents. Referring to a table in the first report of the Rivers Pollution Commission, on pages 28 and 29, we find that in English water-closet towns the total solids in solution in the sewage amount, on an average, to 72.2 parts per 100,000; the organic carbon to 4.696 parts per 100,000; the organic nitrogen to 2.205 parts per 100,000; the ammonia to 6.703 parts per 100,000; the total combined nitrogen to 7.728 parts per 100,000; and the chlorine to 10.66 parts per 100,000. Of the suspended matters the minerals

[1] For a definition of these several elements of the combustion process in detail the reader is referred to Recent advances in water analyses, etc., by the author, in American Monthly Microscopical Journal, May, 1893, p. 127.

amounted to 24.18 parts per 100,000, and the organic to 20.50 parts per 100,000, giving a total of 44.69 parts. If we bear in mind that free ammonia, albuminoid ammonia, and organic nitrogen refer to the same things, and also that the organic nitrogen is usually about double the albuminoid, we may still make comparisons which are close enough for the ordinary purposes of agriculturists.

Comparing several series of analyses, both American and English, it becomes apparent that ordinary town sewage in England is usually considerably more concentrated than that of the American towns. It is very important to bear this in mind in applying English data to American conditions. The elaborate experiments conducted by the Massachusetts State board of health at Lawrence, which have been reported from year to year in the annual reports of that board from 1888 to the present time, indicate that there is a relation between the purifying capacity of different filtrating materials and the amount of impurity which can be removed from sewage of a given strength. This point is strongly brought out by the experiments. It follows, then, that if we prepare special areas for sewage purification in accordance with the indications of the Massachusetts experiments, we may expect to apply sowewhat larger volumes of average American dilute town sewage per unit of area than has usually been found expedient in English practice. If, therefore, we use English data without reference to the quality of the soil to which the sewage is to be applied, or of the sewage itself, we shall be likely to arrive at more or less erroneous conclusions.

As a summation of this part of our discussion, we may say that the chief object of sewage purification is to rid the sewage of a thousandth part, more or less, of the organic matter which it contains, and that all of the appliances for sewage purification and utilization may be considered as directed toward this one point.

POLLUTION OF RIVERS.

This division of the general subject has been discussed in Water-Supply and Irrigation Paper No. 3 under the general head of "Stream pollution." The following additional information is herewith presented by way of extending somewhat the discussion there given:[1]

As an exceedingly marked case of river pollution, we may refer to the report of the Passaic Valley Sewerage Commission, authorized by an act of the New Jersey legislature approved February 26, 1896. The report of the commission appointed under the terms of this act was issued in February, 1897, and therefore may be cited as one of the most recent extended studies on stream pollution to be had.

[1] For a more extended discussion of river-pollution questions the reader is referred to River pollution in the United States, by Charles C. Brown, a paper read before the Engineers' Club of St. Louis, June 18, 1890, and published in the Journal of the Association of Engineering Societies for October, 1890, Vol. X, No. 10, p. 475, and to Sewage Disposal in the United States, Chapter III. The two works give, with the present discussion, the bulk of the published information as to river pollution in the United States.

At its head waters, among the hills of Somerset County, New Jersey, the Passaic River is a pure, clear running stream. The same is true of its principal upper tributaries, the Rockaway and Pompton rivers; and while there is some manufacturing in the upper valleys, the river continues bright and inviting until it reaches Little Falls, at which place it receives the drainage from several residences and factories. After passing Paterson, 3 or 4 miles below Little Falls, its character completely changes, and from that point to Newark Bay, as stated by the Passaic commission, "its pollution is enormous, constant, and increasing yearly." About 436,000 persons now inhabit the district drained by the lower river, furnishing fully 70,000,000 gallons of sewage a day. So great is the extent of the pollution below Paterson that fish life, except a few hardy kinds, has entirely disappeared. The sewage-laden mud shores give out foul odors. Steam users report that the acids of the sewage-laden water have so affected their boilers as to make its use unadvisable. Formerly thousands frequented the river for pleasure purposes, but this is no longer a practice.[1]

In 1880 the population of the towns dischaging sewage into the lower Passaic River was 246,503, and in 1895 it was 436,423.

The ratio of increase in population of the lower Passaic district from 1880 to 1890 is placed at 43.2 per cent, and from 1890 to 1895 at 22.6 per cent. The river now receives the sewage of the cities of Paterson, Passaic, Orange, and Newark, and the towns of East Orange, Bloomfield, Montclair, Harrison, Kearny, East Newark, and part of Rutherford. All these places are growing rapidly, and sewerage facilities for a number of other places are an admitted necessity of the early future. The commission therefore considers it beyond question that the pollution of the lower Passaic is sure to increase enormously unless remedial measures be taken.

As a summation the commissioners state:

(1) That the daily discharge of 70,000,000 gallons of sewage into the Passaic River below the Great Falls of Paterson is beyond its power to assimilate.

(2) That through the stench the pollution has become a nuisance to residents along the banks, and has caused depreciation of property and injury to health.

(3) That fisheries have been destroyed.

(4) That the river has ceased to be desirable for pleasure purposes, boating, bathing, etc., no longer possessing the park value which so generous a stream should afford the large population on its banks.

(5) That the use of the river for manufacturing purposes is suffering on account of the pollution.

(6) In short, that the pollution of the lower Passaic River appears to the commissioners to be completely established as a public nuisance, as injurious to health, and as an increasing menace to property interests *from the Great* Falls at Paterson to below Newark.

---

[1] *Abstract* from report of Passaic Valley sewerage commission, pp. 10-11.

In discussing means of relief the commissioners state that the great progress in sanitary science of recent years has increased the demand for thorough methods of sewage disposal; hence it has been recognized in European countries, and throughout the thickly settled part of the United States, that methods of sewage disposal need a degree of care and thoroughness second only to the requirements of furnishing the same population with water and food. After reviewing the various methods of sewage purification the commissioners conclude that in the present case the conducting of the sewage to tidal waters is, on the whole, preferable. Their reason for this conclusion is, largely, that the available land areas in the Passaic Valley appear poorly adapted for irrigation and filtration processes. On this point the engineers to the commission, Messrs. A. Fteley and Charles E. A. Jacobson, state that the only lands within reach of sufficient area for irrigation works are the extensive meadows of the lower Passaic Valley, but unfortunately the character of the soil is so compact that, even with extensive and costly underdrainage, the amount of sewage which could be disposed of per unit of area would be so small as to require many thousands of acres; hence, Messrs. Fteley and Jacobson consider land disposal impracticable for the lower Passaic. They recommend the construction of an intercepting sewer at an estimated present cost of $6,500,000. The yearly cost of operation is placed at $125,000 in 1900 and $300,000 in 1930.

An extended series of chemical analyses made by Herbert B. Baldwin, chemist to the commission, is included in the report. In considering the effect of the sewage on the condition of the river, it is pointed out by Mr. Baldwin that the average discharge of sewage is a fairly constant quantity, but that the flow of the river is not. By way of illustrating this, we may point out that the flow of the Passaic River varies at Little Falls from a minimum of about 200 cubic feet to a maximum of about 20,000 cubic feet per second. The drainage area above Little Falls is 773 square miles. Whenever the river is either at or anywhere near its minimum flow, what may be termed the point of saturation is so far exceeded that the river inevitably becomes not only a nuisance, but a menace to health. On the one hand, the clearing of forests tends to decrease still further the minimum flow and to increase the length of the low-water period from year to year; on the other hand, the growth of towns and the extension of sewerage works tend to increase the pollution. Taking the two together, there is, therefore, a more rapid increase in actual stream pollution than is indicated by mere consideration of the percentages of increase of the quantity of sewage.

Moreover, the upper Passaic is the natural source of water supply for the north Jersey municipalities, and the probable taking of an appreciable part of the stream for such supplies will inevitably tend to aggravate existing conditions.

Chapter 83 of the Resolves of the Massachusetts Legislature of 1895, directed the State board of health to investigate the sanitary condi-

tion of the meadows on Neponset River, and submit the results of the examination, together with recommendations for the improvement of the sanitary condition of the meadows and the removal of the nuisance therefrom. The report of the board's engineers was made under date of October 1, 1896.

The Neponset Meadows, covering an area of 3,662 acres, of which little more than 600 acres appear to be in a condition adapted to agriculture, are subject to prolonged flooding nearly every year, and, with an increasing pollution of the stream, have become a serious public nuisance. The object of the report of the Massachusetts State board of health referred to is the abatement of this nuisance.

The Neponset River is connected with the Charles River by Mother Brook, which is legally entitled to receive one-third of the flow of Charles River. The drainage area of the Neponset River at its mouth, including one-third of the watershed of the Charles River above Mother Brook, is given as 180.3 square miles. The population of the several towns in the Neponset Basin above the lower end of the meadows was, in 1885, 15,097; in 1890, 17,361, and in 1895, 19,193. These towns are provided with public water supplies, but none of them have as yet a general system of sewerage.

The Neponset River may be divided into three portions: An upper portion, above the great meadows; a middle portion, including the meadows subject to overflow; and a lower portion, within the limits of the Boston metropolitan district.

The upper portion is about 10 miles in length, with a total fall of about 220 feet. Manufactories and mills are located throughout the valley, and the river is highly polluted by manufacturing wastes. The length of the middle portion is 11.25 miles. In much of this portion the river has very little fall and the flowing water is partially purified by dilution with water from purer tributaries and by sedimentation. The lower portion has considerable fall to tide water, and has high banks occupied by factories and dwelling houses. Practically all the water power of this section is made available by milldams. The minimum flow of the stream, as determined by the lowest level, reached in 1895, is about 54 cubic feet per second.

Chemical analyses of the water of the Neponset River were made by the Massachusetts State board of health in 1873, 1875, 1885, 1891, and 1895. A tabulation of the results shows a much more serious pollution in 1895 than in 1875. Thus, in 1875, at the Milton lower mills, free ammonia was 0.0112 part per 100,000; in 1891 it was 0.0274 part; and in 1895, 0.0526 part. In 1875 the albuminoid ammonia at Milton lower mills was 0.0171 part per 100,000; in 1891, 0.0329 part; and in 1895, 0.0342 part. Chlorine at Milton lower mills in 1875 was 0.46 part; in 1891, 1.18 parts, and in 1895, 1.47 parts.

The report states that, in order to improve the sanitary condition of the river and meadows, it will be necessary to prevent the further dis-

charge into the stream of domestic and manufacturing sewage, unless it has been previously purified sufficiently to prevent offense to sight or smell. A system of sewerage was provided for by chapter 406 of the Massachusetts acts of 1895, by means of which it will be possible in future to prevent the pollution of the river below the great meadows, after some provision is made whereby the diversion of sewage from the river into the sewers is made compulsory and the further discharge of manufacturing wastes is prevented.

The town of Hyde Park is located on the Neponset River, just below its junction with Mother Brook. The pollution in the region above at the present time is caused almost entirely by the discharge into the river of manufacturing wastes, generally from paper mills and tanneries. Investigations with reference to the purification of wastes of this sort were begun by the Massachusetts State board of health in the summer of 1895, and a résumé of the results will be found in the following pages. The published results indicate that it will be feasible to purify the manufacturing wastes satisfactorily at a cost sufficiently small to avoid crippling the manufacturing industries. A study of the question of the purification of the sewage of a number of towns naturally draining into the Neponset River in the upper valley shows that the necessary purification can be best accomplished by separate treatment upon land, as is done in many other towns in Massachusetts.

The removal from the river of the crude sewage of the manufacturing establishments, as well as that of the towns in the valley, will tend to improve the sanitary condition of the channel of the stream by preventing further deposition of organic matter upon the sides and bottom and in the numerous shallow bays and coves.

The report further states that, in order to improve the sanitary condition of the meadows, it will be necessary to lower the level of the water in the river sufficiently to permit the satisfactory drainage of the meadows and to prevent being flooded during the summer season. To accomplish this it is considered that the river should be deepened sufficiently to carry off the summer flows and still keep the water surface below the level of the meadows. Applying the run-off data of the Sudbury River watershed, for which a record covering twenty-one years is available, it appears that the river channel should be enlarged enough to enable it to carry at least 720 cubic feet per second, which would be sufficient for anything except a summer rainfall in excess of that of August, 1889, when the run-off of the Sudbury River for the month amounted to 2.55 inches on the watershed, the rainfall of the previous month of July being 8.94 inches and of the previous August 4.18 inches. The run off of August, 1889, of 2.55 inches, was the largest for any summer month during the period 1875 to 1896, covered by the Sudbury measurements. In 1882 the August run-off of the Sudbury was only 0.10 inch. In 1883 the September run-off was 0.08 inch. In 1883 the total run off of the Sudbury for the summer months, June to

August, inclusive, was only 0.86 inch. In 1889 the total run-off of the summer months was 4.81 inches. According to the report of the engineer, an examination of the rainfall records of eastern Massachusetts for the period of thirteen years indicates that summer flows as heavy as in August, 1889, may be expected about once in ten to fourteen years; while for about two or three times in a century it may happen that the flow for some one of the summer months will be considerably in excess of that for August, 1889.

The plan for draining the Neponset Meadows includes, in effect, the cutting of a new channel across a number of sharp bends, thus considerably shortening the length and increasing the slope. It also includes removing the flashboards of the dam of the Mattapan Paper Mill, which is situated on the river about half a mile up the stream from the north line of the town of Hyde Park. The estimated cost of the river improvement, not including cost of land and water damages, is $127,115. It is also estimated that the increase in valuation of the 3,655 acres of meadow resulting from this improvement would amount to $101,330.

In concluding his report, Mr. Goodenough, the engineer of the State board of health, states that the gain in valuation of the meadows, produced by draining them, probably does not represent the full economic value of the improvement. By leaving the river and meadows in their present condition they become not only a source of annoyance, but a menace to the health of those living in the vicinity, and ultimately unfavorably affect the value of all real estate in the vicinity. Moreover, if a portion of the improved meadows should be utilized for market gardening, the value of such portion would be much greater than if used for hay and pasturage alone.

In order to show the great pollution to which streams in manufacturing districts are subject from the discharge of manufacturing wastes, we may further refer to the report on river pollution in Connecticut, by Prof. S. W. Williston, from which extracts have been made in Water-Supply and Irrigation Paper No. 3, first considering the case of Quinnipiac River. That stream has a drainage area of about 150 square miles, with a population thereon in 1890 of fully 40,000. Meriden, the chief city of the drainage area, had a population in 1880 of 15,540, and in 1890 of 21,652.

A measurement of the flow of the Quinnipiac River made August 3, 1888, gave about 7 cubic feet per second. A measurement on Monday, August 6, before the mills started, gave 5.5 cubic feet per second; at 10 a. m. the same day the flow was nearly 20 cubic feet per second. The rainfall is such that in ordinary years we may expect fairly uniform daily stream flows in the vicinity of Meriden.

In 1888 there were in Meriden 23 manufacturing establishments, with 5,708 employees, of whom 3,608 used water-closets.

In a year these manufactories produce the following amounts of *polluting materials:*

Polluting materials used by manufactories in Meriden, Connecticut.

Mineral acids	pounds	891,700
Alkalies	do	198,500
Mineral salts	do	7,000
Inorganic and organic oils	gallons	18,445
Asphaltum	pounds	1,000
Soap	do	7,000
Cotton waste	do	2,000

The organic matter scoured from wool is stated at over 500 pounds daily. The total amount of manufacturing wastes turned into the stream is estimated at from 2½ to 3 tons daily, one-fourth of which is organic.

At Yalesville and Wallingford Professor Williston found 6 manufacturing establishments, with 1,060 employees, of whom 485 used water-closets. The polluting material used per year by the 6 establishments was reported as follows:

Polluting materials used by manufactories in Yalesville and Wallingford, Connecticut.

	Pounds.
Organic matter (about)	50,000
Acids	173,000
Alkalies	41,000
Minerals and mineral oils (about)	90,000
Total	354,000

The sewers of New Britain, Connecticut, drain partly into a tributary of the Quinnipiac River and partly into a tributary of the Connecticut River. The population of New Britain in 1880 was 11,800, and in 1890, 16,519. Professor Williston's canvass showed 12 establishments, with 3,075 employees, using water-closets which drained into Pipers Brook, a tributary of the Connecticut.

The following quantities of chemicals per year were discharged from these 12 establishments:

Polluting materials discharged from manufactories in New Britain, Connecticut.

	Pounds.
Metal salts	700,000
Free acids	100,000
Lime salts	35,000
Alkali salts	100,000
Soap	25,000
Fatty matter	100,000
Vegetable refuse	20,000
Total	1,080,000

Waterbury, the chief city on the Naugatuck River, in Connecticut, had, in 1880, a population of 17,806, and in 1890 a population of 28,646.

In 1887, 26 manufacturing establishments employed 7,571 persons, of whom 7,140 used water-closets.

The annual pollution from manufacturing wastes at Waterbury was placed as follows:

Polluting materials discharged from manufactories in Waterbury, Connecticut.

	Pounds.
Free acids	500,000
Metal salts	2,500,000
Alkaline salts	75,000
Soap	50,000
Fatty matters	200,000
Total	3,325,000

A large amount of interesting recent information in regard to river pollution in the United States is now available, but lack of space precludes further reference to it in this place.[1] The examples given will serve to illustrate the immense amount of polluting wastes that are daily being thrown into the rivers.

SEWAGE PURIFICATION AT MANUFACTURING ESTABLISHMENTS.

In England the pollution of streams from manufacturing wastes became very serious many years ago, and as the result of careful consideration of this special phase of stream pollution a number of sewage-disposal plants have been devised, especially with reference to purifying the sewage of each manufacturing establishment at the mill. A large amount of information in regard to such plants, with detailed plans, may be found in the fourth report of the Rivers Pollution Commission. As American examples, a number of cases cited by Mr. Clark in his report to the Massachusetts drainage commission, together with several recent ones, may be mentioned.

(1) The Wansuch Mills at Providence, Rhode Island, manufacture woolen and worsted goods extensively. The yearly amount of refuse contained in the waste water from this mill is stated to include about 64,000 pounds of dyestuffs, 1,000,000 pounds of alkali, 4,000 pounds of acid, 53,000 pounds of fuller's earth, and 400,000 pounds of grease. The waste water which contains this polluting material, and which originally flowed directly into West River, is stated as about 400,000 gallons a day. A dyeing and bleaching company located on the stream below the Wansuch Mills brought suit, alleging serious injury to its operations. After protracted litigation the supreme court granted a permanent injunction. In compliance with the injunction attempts were made to purify the waste water before permitting it to enter the river. For this purpose an acre and a half of land was prepared for filtration

[1] See the annual reports of the several State boards of health. The annual reports of the Provincial Board of Health of Ontario may also be referred to. As further illustrating the pollution of streams by mill wastes, reference may be made to the Eighty-third Annual Report of the Philadelphia Water Department (1885), pages 308, 309, where the detail of the daily wastes from an extensive carpet, blanket, and cloth mill may be found. The same data are also given on page 64 of Sewage Disposal in the United States. See also the report of the Second Rivers Pollution Commission.

by making furrows 4 feet apart on the surface. This process was fairly successful when first tried, but the land soon became clogged on the surface. The filtration process was then abandoned, although it seems clear from present information that with a larger area it could have been made successful. Purification by chemical precipitation was then adopted and has, so far as known, been continued since. For this purpose a set of six connected basins was excavated on the land previously used for filtration.

It is stated that about a barrel of lime is added to each 100,000 gallons of waste before pumping to the precipitation basins. This addition is made rudely, without either previously grinding or slacking the lime. The mill wastes flow continuously through the basins, with most of the deposition taking place in the first basin. The effluent from the last basin is said to appear to the eye about as foul as when it enters; this apparently indicates, in view of the satisfactory results gained elsewhere, defects in the practical management of the process. In the beginning sulphate of alumina was used as a precipitant, at a cost of about $6,000 per year for the whole amount treated, which, however, was considered too great an expense.

(2) At the Loraine mills, Saylesville, Rhode Island, a method of wool scouring is practiced by which the grease is recovered and most of the dirt is eliminated from the wash water before it is permitted to escape. The results at this mill indicate the recovery of about a ton of grease from each 18,000 pounds of wool washed. The cost of the plant for accomplishing this, not including buildings, was $2,500. The process is considered remunerative.

(3) Two mills in Millbury, Massachusetts, each scouring about 1,000 pounds of wool per day in the grease, retain the first scour in vats, which are cleaned periodically and their contents used as fertilizer. The process is considered remunerative at these mills.

(4) At the woolen mills of Robert Bleakie & Co., Hyde Park, Massachusetts, about 3,000 pounds of wool are scoured daily, the refuse from which, together with sewage and dyeing wastes, flows into a settling basin, whence the effluent passes into the stream. The settling basin consists of a cemented structure 80 feet long, 10 feet wide, and 3.5 feet deep; it is cleaned at intervals and the sludge is used for fertilizer, yielding for this purpose an estimated value of several hundred dollars a year.

(5) At Maxwell's tannery, Winchester, Massachusetts, a mechanical filter is used for straining bark and coarse lime from tannery wastes. The filter consists of a wooden box about 4 feet wide, 2 feet deep, and 60 feet long, divided into compartments filled with hay, through which the waste water filters. The effluent generally is clear, but of a deep mahogany color.

(6) The Nemasket mills at East Taunton, Massachusetts, applied to the State board of health in 1891 for advice with reference to disposing

of the sewage of the mill. The board advised that the waste water from the sinks be kept entirely separate from other wastes and conducted to a tight cesspool with an overflow therefrom into a filtering trench on the bank of the river. Also, that privies be provided in which all excrementaceous matters, both solid and liquid, be retained in water-tight vaults, to be cleaned from time to time, and the deposits buried in the ground not less than 100 feet from the Taunton River. The board expressed the opinion that by carrying out this plan the water supply of the city of Taunton, which is taken from Taunton River below the Nemasket mills, would not be contaminated by the mill wastes.

(7) The Menominee River Valley, in Wisconsin, adjacent to the city of Milwaukee, has been badly polluted for a number of years by wastes from large manufacturing establishments as well as from the public institutions of Milwaukee County. In 1893 the Wisconsin legislature instructed the State board of health to prepare plans and estimates for a sewerage system to serve this district. It also enacted further legislation, requiring all sewage to be filtered or otherwise purified to the satisfaction of the State board of health before being discharged into the stream. The act, however, failed to provide funds for preparing the plans, but most of the manufacturing establishments and public institutions have voluntarily introduced some method of treating their sewage. A recent letter from Dr. U. O. B. Wingate, secretary of the Wisconsin State board of health, indicates that the results obtained have been hardly satisfactory on account of adverse decisions of the Wisconsin courts[1] as to the powers of the State board, which have operated to prevent such supervision of sewage-disposal projects in Wisconsin as is now common in the more advanced States of the East and middle West.

The following are some of the particulars of the works actually constructed. At Schmidt's curled-hair works the solid matter of the liquid wastes is first screened out and the effluent then discharged upon porous soil, where it disappears. A similar system is in operation at McBride's sanitarium.

The National Soldiers' Home, with over 2,000 inmates and a daily flow of sewage of about 270,000 gallons, constructed an intermittent filtration system, the permanent success of which, however, is said to be doubtful because of the character of the soil of the filter areas.

At the shops of the Chicago and Northwestern Railway Company at Merrill Park a furnace has been provided for cremating the excrements.

The public institutions of Milwaukee County and the Miller Brewing Company have constructed combined chemical precipitation and mechanical filtration plants. The county institutions consist of a hospital, almshouse, two insane asylums, etc., having in all over 1,000 inmates. The flow of sewage is placed at from 250,000 to 400,000

[1] See also Sixteenth Ann. Rept. Wisconsin State board of health.

gallons a day. The sewage is first treated with lime and sulphate of alumina, and is subsequently filtered. The works are said to receive daily attention from a chemist, and thus far have furnished a clear effluent, free from odor, which does not undergo a secondary decomposition. However, according to information in Engineering News of January 13, 1895, it appears that this plant is not doing the work as thoroughly as it should because of overcrowding and too infrequent cleaning. The plant was originally designed to treat 480,000 gallons of sewage in twenty-four hours, which would certainly be a very large quantity from 1,000 inmates, even at insane asylums. If the present works actually have a capacity of 480,000 gallons in twenty-four hours, or, what is the same thing, a capacity for treating 480 gallons per capita per diem, they certainly ought to take care of the sewage from these institutions. The mechanical filter is said to be composed of limestone, Florida moss, and iron manganese. We understand that the process is patented, but probably does not represent the best practice in sewage purification at the present time, which may be the real reason for its failing to do the work properly.

The Miller Brewing Company, which produces perhaps 150,000 gallons of sewage a day, has a similar plant, the effluent from which is stated to be so unsatisfactory as to undergo secondary decomposition.

In a letter to Engineering News dated January 21, 1895, Dr. Wingate, the secretary of the Wisconsin State board of health, states that two of the plants which have been built in the Menominee Valley are satisfactory in their construction, but their management is unsatisfactory. He also states that the managers of the different institutions and manufacturing establishments have apparently the idea that constructing a plant is all that is necessary, that they have not furnished the necessary agents and help to keep the plants in proper condition, and that they have not managed them properly.[1]

Experience abroad has fully shown that purification works require attention; indeed, our own experience on this point is now so full, that it seems unnecessary for Americans to experiment further in that direction.

(8) In Ohio stream pollution from manufacturing wastes has assumed some importance, and the State board of health of that State has investigated a number of cases of such pollution, one of which may be referred to here. The Scioto Straw-Board Company, of Kenton, Ohio, discharges the wastes from its mill into the Scioto River. Complaints having been made to the State board of health that such discharge caused a public nuisance, an investigation was made in the summer of 1886. The following analyses of water from the Scioto River at Kenton indicate the extent of the pollution. In considering the significance of

[1] In a letter to the author, dated August 6, 1897, Dr. Wingate states that a drainage commission has been appointed to report to the next legislature, which will consider the whole question of Menominee Valley sewage disposal.

these analyses it should be understood that the Scioto River at Kenton is little more than a respectable brook. The average fall does not exceed 1 foot per mile for 30 miles. Analysis No. 1 is water from the Scioto River above the mill; No. 2, water from below the mill, and No. 3, from La Rue, a few miles below.

Analyses of water from Scioto River near Kenton.

[Grains per gallon.]

Number.	Lime.	Chlorine.	Mineral.	Organic and volatile.	Total solids.
1	8.98	0.25	20.30	9.68	29.98
2	63.32	6.49	84.09	130.63	214.72
3	13.33	1.45	31.26	18.19	49.45

So far as can be learned, the great increase in pollution of the stream did not affect the health of people living in the vicinity, although the fish in the stream were killed and there was at time of low water a serious effluvium nuisance. The remedy was the construction of a settling tank of from 3 to 5 acres. As to whether or not this has proved satisfactory there is no information at hand.[1]

(9) The waste waters at Tid's tannery, Stoneham, Massachusetts, were treated chemically during the years 1894 and 1895. The following results were obtained in the latter year. The total quantity of sewage pumped was 4,990,924 gallons, to which were applied as a precipitant 71,760 pounds of sulphate of alumina. The sludge amounted to 648,820 gallons, or 13 per cent of the sewage. It was disposed of to neighboring farmers, who carted it away for fertilizer. These works were first operated in 1893, by the city of Boston, in order to protect the Upper Mystic Lake.

(10) At Fitzgerald's tannery, also in Stoneham, Massachusetts, a series of precipitation tanks were constructed by the owner in 1895, arranged in such manner that the heavy particles of waste matter will settle as the sewage flows from one tank to another over separating partitions. In 1895, however, the quantity of sewage having increased to about 10,000 gallons daily in August, it was found necessary to use more chemical than the lime naturally present in the sewage. The owner thereupon constructed a vat for chemicals, and the Boston water department furnished and applied sulphate of alumina to the extent of 10,520 pounds from August, 1895, to January, 1896. The effluent from both these tanneries in Stoneham, while still highly colored from the tan bark, is stated to be on the whole fairly satisfactory. The use of a larger amount of chemicals would remove an additional amount of color, but at such increased cost as is considered not to be justified by the circumstances.

[1] *The Ohio State board of health issued in 1898 a Preliminary report of an investigation of rivers, etc., of Ohio, which gives much information about the waters of that State.*

MASSACHUSETTS EXPERIMENTS ON THE PURIFICATION OF FACTORY WASTES.

In the report of the Massachusetts State board of health for 1895 the results of investigations as to the disposal of waste liquors from various industrial processes are given. Among others may be found a discussion of the results obtained in the purification of the waste liquors from paper making, wool scouring, and tanning.

PAPER-MILL WASTES.

The experiments on purification of paper-mill wastes included the application for some time to a filter containing 60 inches in depth of sand of the effective size of 0.25 millimeter of the waste liquor resulting from cleansing rags in a paper mill by means of a boiling solution of caustic soda and lime. The rags cleaned are of all kinds and colors, and the resulting liquor, as stated, is very highly colored. It was applied to the filter at an average rate of 65,500 gallons per acre daily, with the result of great improvement in the appearance and analysis, although the effluent as it flows from the filter is still highly charged with organic matter in solution. The following is the analysis of the applied liquor: Free ammonia, 2.3 parts per 100,000; albuminoid ammonia, 4.1 parts; chlorine, 20 parts; oxygen consumed, 140 parts. The effluent shows: Free ammonia, 1.8 parts per 100,000; albuminoid ammonia, 2.29 parts; chlorine, 12.59 parts; nitrogen as nitrates, 0.105 part, and as nitrites, 0.0067 part; oxygen consumed, 84.36 parts. The nitrates and nitrites in the applied liquor are nil. It is clear, therefore, that some considerable nitrifying action has taken place in the filter, but it is the opinion of the board that the degree of alkalinity is so high as to prevent the full effect of nitrification.

WOOL-SCOURING WASTES.

The waste liquors of wool scouring were applied to a filter with the same depth of sand and effective size of grains as in the previous case, at the rate of 17,000 gallons per acre daily. The results, so far as obtained, show that while a considerable improvement of such a liquor can be obtained by intermittent filtration, still, owing to clogging of the upper layers of the sand, it appears desirable that a preliminary treatment, to remove a part of the dirt and fatty matters before filtration, should be applied. The average analyses of the waste liquor from wool washing and of the effluent were as follows: The applied liquor contained 17.3 parts of free ammonia per 100,000; albuminoid ammonia, 43.1 parts; chlorine, 60.13 parts; oxygen consumed, 232 parts. The effluent contained: Free ammonia, 27.8 parts per 100,000; albuminoid ammonia, 8.3 parts; chlorine, 47 parts; nitrates, 0.28 part, and no nitrites; oxygen consumed, 90 parts. The applied liquor was entirely free of both nitrates and nitrites. A refiltration of the effluent

from the preceding experiment on waste liquor from wool scouring was made through two filters, one containing 60 inches in depth of sand of the same effective size of grain as before and the other containing 60 inches in depth of coke breeze. The effluent was applied to the two filters for three months, at the rate of 50,000 gallons per acre daily. The liquor passed through both the sand filter and the coke breeze very little changed, thus showing not only the stable character of the organic matters present, but that the high degree of alkalinity was probably prejudicial to nitrification.

An experiment was also made as to the treatment of the waste liquor from wool scouring by the preliminary precipitation of the fat and dirt by means of calcium chloride, followed by intermittent filtration. This treatment gives an almost complete clarification of the liquor, but it passed through a filter 5 feet in depth, with effective size of sand grain of 0.25 millimeter, with very little change.

TANNERY WASTES.

In order to test the efficiency of intermittent filtration as applied to tannery wastes, an experimental filter has been set up at Norwood, Massachusetts. On account of the conditions imposed by the location, the filter has only 2 feet of sand, the effective size of the sand grain being 0.14 millimeter. The sewage applied is a mixture of the waste liquors from all the processes carried on in the tannery; and it was applied to this shallow filter at an average rate of 55,000 gallons per acre daily. The results obtained were as follows: The applied sewage contained 3.1 parts of free ammonia per 100,000; albuminoid ammonia, 3.16 parts; chlorine, 312 parts; nitrates, 0.16 part; nitrites, 0.0015 part; oxygen consumed, 55 parts. The effluent contained: Free ammonia, 2.34 parts per 100,000; albuminoid ammonia, 0.6 part; chlorine, 290 parts; nitrates, 1.05 parts; nitrites, 0.01 part; oxygen consumed, 7.5 parts.

This sewage is said to be very strong and offensive, containing decaying animal tissues and the bran added to cause fermentation. It is also colored by the dyes used in finishing the hides, which, after a few weeks, the filter failed to remove completely. The applied sewage did not contain any amount of the spent tan liquor. The antiseptic qualities of the tannic acid would, without doubt, be very prejudicial to nitrification.

In cleaning hides a large amount of lime is used, in consequence of which in large tanneries there is an almost continuous stream of lime water flowing into and mixing with the other waste liquors. By properly arranged settling tanks and the regulation of this lime water, it is the opinion of the Massachusetts board that a large percentage of the sludge could be precipitated out. Experiments show that probably 60 per cent of the organic matter can be removed in this way, although even then the resulting supernatant sewage would still remain very rich in organic matter in solution.

The supernatant sewage from such treatment was applied to a small filter at the average rate of 120,000 gallons a day. After several months' operation this filter is stated to be in a state of active nitrification, giving a clear and nearly odorless effluent. It is the opinion of the Massachusetts board that by treatment of tannery sewage on this line it may be efficiently purified by filtration.

DYESTUFFS.

Among the many experiments carried out at Lawrence, perhaps as interesting as any, are those relating to the filtration of sewage containing dyestuffs. This division of the sewage question is specially interesting because of the large number of manufacturing establishments at various places, the refuse from which usually passes without treatment into streams. In England this phase of sewage purification became of considerable importance early in the present century, because, generally speaking, the streams of England are small, and even sixty years ago a number of them had become badly polluted by the waste dyes from woolen, cotton, and other fabric manufacturing establishments. As illustrating the conditions twenty-five years ago, the Second Rivers Pollution Commission gives in its third report a reproduction of a memorandum written with the water of the river Calder at a point where the Wakefield sewer enters that stream. This memorandum reads as follows: "Dedicated without permission to the local board of health, Wakefield. This memorandum written with water taken from the point of junction this day, between the river Calder and the town sewer. Could the odor only accompany this sheet it would add much to the interest of this memorandum."

The condition of the river Calder has been improved in recent years by the construction of a number of sewage purification works along its banks, although when seen by us in October, 1894, it was still far from being a limpid stream.

Without going into the detail of the Massachusetts experiments on the filtration of various dyes used in fabric manufacturing, we may simply state that indigo extract, logwood extract, alizarin base, archil B, rose azurine, benzo-azurine, methyline blue, orange 2 R, magenta, malachite green, and scarlet 2 R, when applied at varying rates—from 335,000 gallons per acre of archil B to 1,280,000 gallons per acre of methyline blue—gave an effluent absolutely free from color. Scarlet 2 R applied at the rate of 1,076,000 per acre gave an effluent as highly colored as the applied sewage, but when applied at the rate of 720,000 gallons per acre the effluent was without color. Patent blue, when applied at the rate of 56,000 gallons per acre, gave a slight color; it also gave a slight color when applied at the rate of 160,000 gallons per acre, but when applied at the rate of from 335,000 gallons per acre to 960,000 gallons, it gave, in each case, a strongly colored effluent.

Indigo extract, logwood extract, alizarin base, and archil B are dilute

colors. The others are coal-tar colors in concentrated form. In most of the experiments the removal of the color, as already stated, was complete, as tested by viewing the effluents in tubes of from 200 to 600 millimeters depth.

Considering the large quantities of dyestuffs applied in these experiments, the conclusion may be drawn that dyes as likely to exist in ordinary city sewage are easily amenable to filtration treatment.

SPECIAL EXPERIMENTS.

In the course of the experiments at Lawrence a number were made as to the effect of antiseptics, as well as other substances, upon nitrification. Some of the more important of these experiments may be briefly referred to.

(1) For several weeks a considerable quantity of egg albumen, a substance nearly insoluble in water, was applied in one of the experimental tanks, the object being to determine to what extent it would be rendered soluble and converted into free ammonia. The indications of the first experiment were that about 61 per cent of the total nitrogen contained in the albumen applied was rendered soluble and converted into nitrates. A repetition indicated that possibly 90 per cent of the total nitrogen of the albumen was converted into nitrates.

(2) To one of the experimental tanks, which at the time of the first application was giving a perfectly nitrified effluent, a solution of ammonium chloride, in water containing 1 part of ammonia per 100,000, was applied in place of the sewage. Enough sodium carbonate was mixed with the ammonium chloride solution to combine with the chlorine of the ammonium chloride, and also with the nitric acid equivalent to the ammonia. Nitrification was complete from the first, the effluent being not only free from ammonia, but containing nearly all the nitrogen applied, as nitrates. The strength of the solution was gradually increased until it contained 34 parts per 100,000 of ammonia, after which complete nitrification was not at once obtained, but finally a nearly complete nitrification resulted.

(3) Experiments as to the the effect of an excess and deficiency of alkali were also made, showing that with alkali in excess nitrification was not in the least interfered with, but with a deficiency the process apparently stopped with the production of an enormous quantity of nitrites.

(4) Experiments as to the effect of acid upon nitrification were made, which showed that sewage containing a large percentage of sulphuric acid may have a major part of its nitrogenous constituents removed for a considerable time. If we are dealing with a sewage which contains moderate quantities of sulphuric acid occasionally, we may expect to purify it by intermittent filtration without any ill effects; but if the sewage contains sulphuric acid regularly, the acid should be neutralized *by the addition* of lime or some other alkali.

(5) Experiments as to the effect of common salt showed that when common salt was present in such quantity that the chlorine amounted to 1,200 parts per 100,000 the nitrification was quickly checked and the common-salt solution passed through the filter almost unchanged. If, however, common salt be added gradually, so that the filter may have the opportunity to adapt itself to the work required of it, the result was found to be very different. In an experiment of this character it was ascertained that with gradual increase of the common-salt solution nitrification continued. The practical result arrived at was that sewage containing a considerable quantity of common salt as one of its regular constituents may be purified by intermittent filtration without difficulty.

(6) Experiments as to the effect of sugar upon nitrification showed that if a considerable quantity of sugar is applied to intermittent filtration it will cause a decrease of nitrification. If, however, the filter can be gradually adapted to the special work of nitrifying sugar, the nitrification will finally become nearly as complete as when sugar is absent.

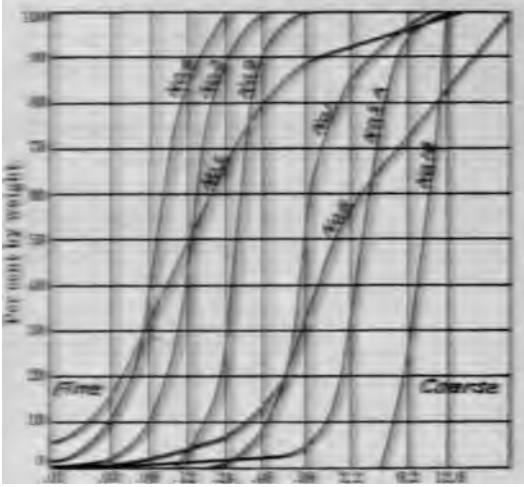
MECHANICAL ANALYSES OF SANDS.

The selection of the filtering materials is a very important point in constructing a filter. The Massachusetts State board of health has developed a method of sand analysis by which different sands may be compared and from their relative coarseness their efficiency as a filter material be foretold.

SIZE OF GRAIN.

As described in the twenty-third annual report of the board, the sand is first sifted through a series of sieves, each about twice as fine as the one next coarser. The sand passing the finest sieve is divided into several portions by beaker elutriation. Each portion is weighed, and the range and size of particles is determined either by micrometer measurement of the smaller particles or by computation of the diameters of larger particles by their weight. The diameters of all particles are taken as nearly as possible at the diameter of a square of equal volume. In each case the diameters are expressed in millimeters. As illustrating the results of such mechanical analyses in their application to filter material, we may quote the following as the percentage statement of the fine material used in experimental filter No. 5. In this filter the material with mean diameter less than 12.6 millimeters amounted to 99 per cent of the whole; material with less diameter than 6.2 millimeters to 96 per cent of the whole; mean diameter less than 2.2 millimeters, 92 per cent; mean diameter less than 0.98 millimeter, 89 per cent; mean diameter less than 0.46 millimeter, 80 per cent; diameter less than 0.24 millimeter, 67 per cent; diameter less than 0.12 millimeter, 51 per cent; diameter less than 0.06 millimeter, 33 per cent; diameter less than 0.03 millimeter, 16 per cent; diameter less than 0.01 millimeter, which would represent the organic matter, 6 per cent.

We have an example of the mechanical analysis of a coarse sand in the case of sand from filter No. 1. In this filter the material with mean diameter less than 12.6 millimeters amounted to 100 per cent of the whole; material with diameter less than 6.2 millimeters, to 97 per cent of the whole; mean diameter less than 2.2 millimeters, 85 per cent; mean diameter less than 0.98 millimeter, 53 per cent; mean diameter less than 0.46 millimeter, 7 per cent; mean diameter less than 0.24 millimeter, 1.5 per cent; mean diameter less than 0.12 millimeter, 0 per cent.



Diameter of sand in millimeters.

FIG. 1.—Mechanical composition of materials available for sewage filtration.

In the same way the results of the mechanical analysis of other sands used in the Lawrence experiments are expressed in the reports.

In order to compare the results of the mechanical analyses of the several sands experimented with as well as their relative values for filtration, we include the accompanying diagram, fig. 1, derived from the Massachusetts reports. The lines representing the diameters are spaced according to the logarithms of the diameters of the particles, as in this way materials of corresponding uniformity in range of size of particles give equally steep curves, regardless of the absolute size of the particles. The materials indicated by fig. 1 are stated by Mr. Hazen to include the whole range of the sands available for sewage purification.

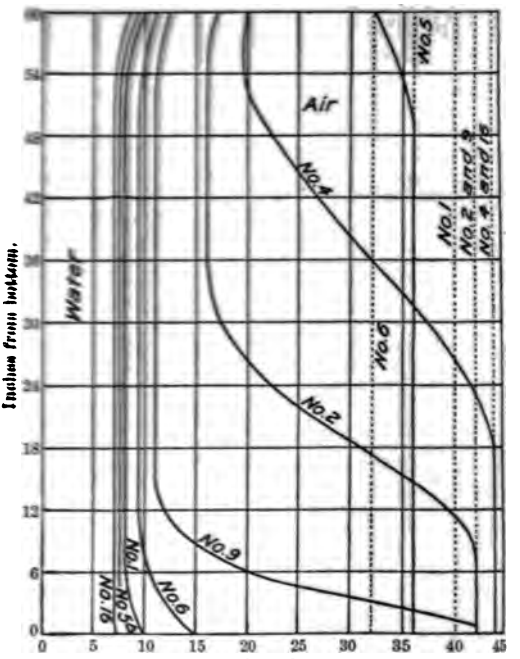


FIG. 2.—Air and water capacities of sewage filter materials. Full lines represent water capacity and dotted lines air capacity.

AIR AND WATER CAPACITIES OF SAND.

rder to estimate the filtering capacity of any given material, it is tant to understand the air and water capacity of the filtering ial when drained, the term "water capacity" being taken to des- the amount of water retained in the interstices after thorough ng. It is clear from what has preceded that the efficiency of the ıg process must depend largely upon these two elements. In also derived from the Massachusetts reports, the air and water ities of the same materials illustrated by fig. 1 are shown, though dying this diagram it should be borne in mind that the curves for ıce and water capac- n be taken only as ıl averages, because ınacity of the mate- ın be varied greatly king. It is obvious he amount of water epend not only upon oseness of the pack- ıut also upon its mity, the tendency s being toward an se of the water ca- . It will also depend given time upon the ıt of organic matter . from the material d. The water capac- l also depend largely the size of the particles, the finer sands holding much water, espe- at the bottom, while with coarse sands the amount held will be constant from top to bottom.[1]

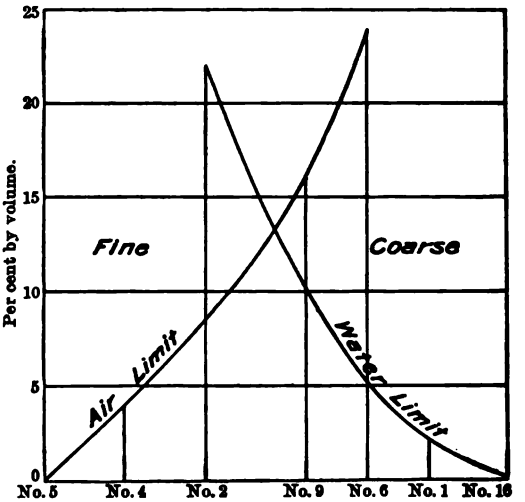


FIG. 3.—Limitation of size of single dose by air and water capacities.

LIMITATION OF SIZE OF DOSE.

diagram, fig. 3, also taken from the Massachusetts reports, will ate the range of capacity with different materials in this partic- This figure illustrates the limit of size of a single dose with refer- o air and water capacities. The various materials are indicated rtical lines, while the per cent of total volume is indicated by ıntal lines, the lower portion of the two curves showing the maxi- dose for any given material which can be applied at one time good results. If the dose is greater than that indicated in the of water limit, it is probable that some will pass the filter before

[1] ence to the twenty-third annual report of the Massachusetts board will furnish a discussion important practical questions in full detail.

complete purification. If the air limit is exceeded, the oxygen in the filter is liable to be exhausted before the oxidation is complete.

In the line of these studies the Massachusetts reports give a table showing the doses proved to be adapted to the various materials under the most favorable conditions of management. For material No. 16 of the preceding diagrams, as actually applied in practice at the experiment station, the dose was 2,800 gallons per acre, applied 500 times a week, this amount giving a daily average of 200,000 gallons per acre. For material No. 1 the size of dose is 40,000 gallons, applied 18 times a week, giving an average daily amount of 103,000 gallons per acre. For material No. 6 the size of dose is 70,000 gallons per acre, applied 6 times a week, giving a daily average of 60,000 gallons per acre. For material No. 9 the size of dose is 120,000 gallons an acre, applied 6 times a week, giving a daily average of 103,000 gallons per acre. For No. 2 the dose is 140,000 gallons an acre, 3 times a week, giving a daily average of 60,000 gallons per acre. For No. 4 the dose is 80,000 gallons, 3 times per week, giving a daily average of 34,000 gallons per acre. The depth of the material to which these applications were made was in every case, with the exception of No. 6, 5 feet; for No. 6 the depth was 4 feet.

As stated in the report, it must be borne in mind that the above figures are applicable only to clean materials under favorable conditions, and that in order to apply large doses permanently with good results the surface material must be occasionally renewed, as already discussed.

PURIFICATION OBTAINED BY INTERMITTENT FILTRATION.

As to the degree of purification obtained by intermittent filtration, the Massachusetts experiments show that very high degrees are reached easily. By way of illustrating the matter we may simply cite that city sewage has been purified to the extent of removing from 95 to 99.5 per cent of the polluting material. Sewage containing from 500,000 t 1,000,000 bacteria per cubic centimeter has been experimentally so fa. purified of bacteria that the effluents have frequently contained as few as from 25 to 100 bacteria per cubic centimeter. In order to appreciate this degree of bacterial purification we may consider that well waters in common use frequently contain from 2,000 to 3,000 bacteria per cubic centimeter. As regards chemical and biological considerations there is therefore no reason why such sewage effluents are not fit to drink.

SUMMARY OF MASSACHUSETTS EXPERIMENTS.

The body of information in regard to sewage purification by intermittent filtration which is presented in the several reports of the Massachusetts State board of health has become so extensive that one must be an expert to keep in mind the many interesting and valuable results brought out. As assisting the general reader, who may not care to travel through the several thousand pages of matter given in these reports, reference may be made to the very excellent summary of

the results obtained from intermittent filtration during seven years' experimentation, which appears on page 497 of the report of 1894, where may be found brief statements of yearly averages of the various kinds of filters experimented upon, as well as an outline of the most important features in the operation of each filter.

In concluding, it is pointed out that—

(1) The qualitative efficiency of the filters is lower in winter than in summer chiefly because of the inactivity of the nitrifying organisms when exposed to low temperature. The indications at present are that it is not advisable to allow exposed filters to rest in winter, even for limited periods.

(2) Qualitative deterioration is a serious matter in winter, because nitrification can not be promptly reestablished as in summer.

(3) It has been learned, however, that when nitrification is active at the beginning of winter it may by proper treatment be preserved during the cold season. To this end frost should not be allowed to penetrate into the interstices of the filters.

(4) The warmer the sewage, within limits, the better the results. Where high degrees of purification are imperative in winter the sewage may be heated to a temperature of 100° F. We may conclude, then, that the arrangement of the filter bed should be with reference to preserving, so far as possible, the heat actually present in the sewage as it arrives in the filter. For this purpose the trenching of the filter area is especially efficient, because it concentrates the heat, thus aiding in preserving the biological process in a state of efficiency.[1]

(5) In any case sewage should be so applied as to penetrate all portions of the filter, as otherwise the passage through will be rapid and irregular, thus tending to a low degree of purification.

(6) The composition of sewage, particularly as regards the kind of sewage, is a much more marked factor in winter than in summer. This is true even in the case of experimental filters, where special lines of treatment to keep the filters in operation are feasible.

VALUE OF COMMERCIAL FERTILIZERS.

In order to indicate the theoretical value of the nitrogen, phosphates, and potash of crude sewage, the following statement of trade values of fertilizing ingredients in raw materials and chemicals of commercial fertilizers, as used by the agricultural experiment stations of the country, is included. The valuations obtained by the use of these figures agree fairly well with the average retail price of standard materials. The price per pound, in cents, is as follows:

Value of chemical ingredients in commercial fertilizers.

	Cents.
Nitrogen in ammonia salts	17
Nitrogen in nitrates	15.5
Organic nitrogen in dry and fine-ground potash, meat, and blood, and in high-grade mixed fertilizers	17.5

[1] *For an estimate in detail of the cost of heating sewage see Rafter and Baker's Sewage Disposal in the United States, p. 336.*

	Cents.
Organic nitrogen in cotton-seed meal and castor pomace	16.5
Organic nitrogen in fine-ground bone and tankage	15
Organic nitrogen in fine-ground medium bone and tankage	12
Organic nitrogen in medium bone and tankage	9
Organic nitrogen in coarse bone and tankage	7
Organic nitrogen in hair, horn shavings, and coarse potash scraps	7
Phosphoric acid, soluble in water	6.5
Phosphoric acid, soluble in ammonium citrate	6
Phosphoric acid in fine bone and tankage	6
Phosphoric acid in fine medium bone and tankage	5
Phosphoric acid in medium bone and tankage	4
Phosphoric acid in coarse bone and tankage	3
Phosphoric acid in fine-ground fish, cotton-seed meal, castor pomace, and wood ashes	5
Phosphoric acid in fine-ground rock phosphate	2
Potash as high-grade sulphate in forms free from muriates (chlorides), in ashes, etc	5.5
Potash in kainit	4.5
Potash in muriate	4.5
Organic nitrogen in mixed fertilizers	17.5
Insoluble phosphoric acid in mixed fertilizers	2

Value of fertilizing ingredients in foods.

	Cents.
Organic nitrogen	17.5
Phosphoric acid	5
Potash	5.5

NECESSITY OF PURIFICATION OF SEWAGE AT TOWNS ON THE GREAT LAKES.[1]

According to statistics of the Eleventh Census, the basin of the Great Lakes, especially the area contiguous to Lakes Ontario, Erie, and Michigan, is, next to the Atlantic seaboard, the most densely populated area of the United States. Bordering on Lakes Ontario, Erie, and Michigan are great and rapidly growing cities from which all of the sewage now passes into these lakes. The surrounding regions are usually not greatly elevated above the lakes, and hence do not afford any opportunity for obtaining upland waters for municipal supplies. The water supplies of towns on the Great Lakes are not only now almost universally taken from these bodies of water, but must necessarily continue to be so taken in the future. At present the sewage of the Great Lake cities is discharged without treatment into the same bodies of water from which the public water supplies are taken. An exception is the city of Rochester, which wisely brings its water supply by gravity from Hemlock, an inland lake about 30 miles distant; but Buffalo, Erie, Cleveland, Detroit, Chicago, Milwaukee, and many other towns take their water supplies in the manner stated. Without going into details for all the Great Lakes, we will state that Lake Michigan alone receives at the present time the sewage of municipalities and small towns aggregating over 2,000,000 people, and this population is rapidly

[1] This chapter is presented to illustrate the vast importance which sewage-purification studies are rapidly assuming in the United States.

ıcreasing, having about doubled in ten years. A similar increase in opulation has taken place in the cities and towns tributary to the ower Great Lakes. Thus, Detroit had a population in 1880 of 116,340, nd in 1890, 205,876. Cleveland showed a population in 1880 of 60,146, and in 1890, 261,353. In 1880 Buffalo had 155,134; in 1890, 55,664.

EFFECT OF SEWAGE ON LAKE WATERS.

The modern studies all indicate that when organic matter in increas. ıg quantities is continually poured into a body of fresh water, a point quickly reached beyond which the body of fresh water. whether it e lake, pond, or running stream, has no power of further assimilation. hemical analyses of the waters of the Great Lakes show a gradually creasing contamination during the last fifteen or twenty years. The idence is multiplying that, with the present increase in population, e Great Lakes, if they continue to be used as common sewers, will soon come totally unfit for use as drinking water. When this time arrives ıe of two alternatives must be followed—either every source of water pply must be filtered or the sewage of the towns must be efficiently ırified before it is allowed to flow into the lakes. In some cases the nditions are such that both remedies must be applied.

The question of the limit of permissible contamination of these waters very likely to arise, either as to the number of bacteria, the micro- opical forms, or the chemical constituents. The answer is, it will pend entirely upon the environment. If a given water is known to ceive sewage, a very small quantity of organic matter, indicated ther by the free and albuminoid ammonias or by the loss on ignition, ay justly cast suspicion upon it. If, on the other hand, there is no wage contamination and the organic matter present is clearly shown be due to natural drainage from the surface of the ground, the nmonias and loss on ignition have very little significance. As to the ımber of bacteria, Dr. Koch, the originator of the method of gelatin ıltures, laid down the principle that potable waters might show, when amined immediately on collection, as high as 1,000 bateria per cubic ıntimeter without condemning them. The more recent studies, how- er, tend to show that Koch's original figure was somewhat too high, ıd at present it is considered that a sanitarily unobjectionable water ıght not to show more than 300 to 500 forms per cubic centimeter. hen the number rises much above that amount the water is open to ıspicion.

Bacteriological examinations of the water of the Great Lakes show ıat the water at Detroit is superior for domestic use to that at Buffalo. he difference must be ascribed to the vast amount of sewage poured to Lake Erie, and the incapacity of that body of water to purify itself ıoroughly before reaching the Buffalo intake at the head of the iagara River.

If space permitted, many analyses of the water of the Great Lakes might be quoted which show, on the whole, a gradually increasing contamination.

One point of interest about the water supply of the city of Milwaukee may be noted. During the navigation seasons of 1892, 1893, and 1894 the United States Weather Bureau carried out an extended series of float experiments in order to determine the actual trend of the currents of the Great Lakes. As regards Lake Michigan, the experiments clearly show that there is a rapid whirl at the south end, with a strong current passing north along the east shore, while along the west shore the current flows south. This disposition of the currents undoubtedly favors the water supply of the city of Milwaukee, giving to that city water with less organic contamination than it would receive, either if there were no current or if the current set from the south to the north along the west side, bringing to Milwaukee the contaminated water of the Chicago lake front. As the currents now flow Chicago receives the contaminated water of Milwaukee.

At Chicago the barbarous practice of turning the sewage of a great city into the body of water from which the public water supply is taken has resulted so disastrously as to lead to the construction of what is known as the Chicago drainage canal, by which it is expected to deliver the greater part of the Chicago sewage into the head waters of the Illinois River, and so finally into the Mississippi. Partial investigations, made several years ago under the auspices of the Illinois State board of health, [illegible] a very rapid purification of the sewage-contaminated waters of the present Illinois and Michigan Canal as they flow to the south. The investigation has not been carried far enough to indicate what the results will be upon the Illinois River and the Mississippi when the project shall have been completed and the sewage of Chicago largely turned south. The known facts as to the increasing contamination of the Great Lakes would indicate the vast importance of determining the [illegible] effect of such discharge. In considering such effect, [illegible] the difference between running streams and lakes with only slight currents.

DANGER OF WINTER TYPHOID

As regards water supplies from the Great Lakes at places so far north as to render [illegible] lakes by ice nearly every winter [illegible] with taking water either [illegible] sewage, which has thus far been [illegible] lakes remain open [illegible] microbic life will be [illegible] contamination, to keep [illegible] but as soon as the lake surfaces [illegible] new set of conditions

[illegible] Great Lakes in which [illegible]

exist; the air is then excluded and the anaerobic forms develop at the expense of the aerobic. This undoubtedly explains the severe epidemics of winter typhoid which have occurred at Buffalo, New York; Lorain, Ohio; Windsor, Ontario; Ashland, Wisconsin; Duluth, Minnesota; Chicago, Illinois, and at a number of other places on the Great Lakes. The following are some of the particulars of these winter outbreaks:

In the latter part of February, 1894, on account of low water in the Niagara River, it became necessary to take the water supply of the city of Buffalo from an old inlet said to be in a current carrying the discharge from sewers higher up. From March 6 to 11, inclusive, the total number of typhoid cases reported was 36. On March 12, 26 new cases were reported. The next day the number was increased 100, and then by 88, 79, 52, and 25 on successive days, making a total of 406 cases from March 6 to 17, inclusive, and 370 cases in the six days from March 12 to 17, inclusive. These cases were mostly of such mild character that only 9 deaths were reported for the whole month. In March, 1893, only 4 deaths occurred, and in March, 1892, but 1.

The city of Lorain, Ohio, is located at the mouth of the Black River, which receives the sewage of Elyria, several miles distant. It also receives the sewage of Lorain, in which place sewers were first laid down in 1892, their use for water-closet drainage beginning in October or November of that year. The water supply of the town is taken from very near the end of the harbor jetties, where the sewage first has free access to the lake water. Lake Erie was closed with ice at Lorain during the months of January, February, and March, 1893. In March and April there was a serious epidemic of typhoid fever, 8 deaths occurring in a population of about 5,000. In 1895 the lake was again closed with ice during the months of February and March, and typhoid fever again became epidemic, there being 3 deaths in February, 2 in March, and 4 in April.

In February, 1896, typhoid fever became epidemic at Windsor, Ontario, a town on the Detroit River just opposite the city of Detroit. During the previous month of December, 1895, the total number of cases was 9; for January the total was 18; for February 82, and for March 13; making a total for the four months of 122. The cause of the outbreak is ascribed to the presence of the sewage of Walkerville in the Windsor water supply. The discharge of Walkerville sewage above the Windsor waterworks intake has been a source of controversy between Windsor and Walkerville for several years, and although more or less typhoid fever has existed at Windsor the outbreak in February, 1896, is the most serious winter epidemic thus far experienced there. As stated in the report of the Provincial Board of Health of Ontario, there was a severe freshet about January 25, 1896.

During the winter of 1894 a severe epidemic of typhoid fever occurred at Ashland, *Wisconsin.* Several hundred cases developed in a few *weeks, and from the distribution* there was every reason to ascribe

their origin to the condition of the city water supply, which is taken from Chequamegon Bay, an arm of Lake Superior, on which the city is situated. This bay is about 12 miles long, with an average width of 5 miles. It varies in depth from 8 to 36 feet. A large breakwater has been constructed to the northwest of the city, on the city side of which is located the mouth of the intake pipe of the public water supply, about a mile from the shore. This arrangement is unfortunate, because it permits the sewage and storm water of the city to flow directly over the mouth of the intake pipe.

In February, 1894, the ground was covered with 2 feet of snow. In the last week of the month there were heavy rains and warm weather, which in a few days melted all the snow and washed the accumulations of the winter into the bay. Typhoid fever broke out in from ten to fifteen days thereafter, and in a week after the first case made its appearance there were over 300 cases. The epidemic lasted about eight weeks, and finally terminated with 450 cases and 26 deaths. Bacteriological examinations of the water showed that it contained as high as 36,000 bacteria to the cubic centimeter. Typhoid germs were found in one examination, and the bacillus coli communis in every examination made. This experience has led the city of Ashland to construct a sand filtration plant.

A severe epidemic of typhoid fever occurred at Duluth, Minnesota, a few winters ago under conditions similar to those at Ashland.

We have noted the insanitary conditions at Chicago on a previous page. By way of illustrating the prevalence of typhoid in that city we may refer to the following table, showing the number of deaths from that disease in Chicago from January, 1890, to December, 1893, inclusive, taken from the annual reports of the Chicago department of health:

Number of deaths in Chicago resulting from typhoid fever.

Month.	1890.	1891.	1892.	1893.
January	53	67	311	41
February	136	61	187	39
March	108	71	76	41
April	45	136	56	58
May	32	408	70	56
June	107	167	55	60
July	86	200	211	55
August	115	182	179	76
September	85	198	138	86
October	72	171	92	81
November	67	150	67	43
December	47	186	47	43
Total	1,008	1,987	1,480	670

The above table shows that typhoid fever was epidemic in Chicago in February and March, 1890, in April, May, June, and July, 1891, and in January and February, 1892. All these outbreaks, as well as the general high typhoid death rate, must be attributed entirely to the polluted water supply. The next year, 1893, was the year of the Columbian Exposition, and much greater efforts were made in that year to insure proper sanitary conditions in Chicago. That these efforts were well directed is shown by the great reduction in the typhoid death rate for that year.

In regard to typhoid fever, it may be assumed as axiomatic that it is a disease of the fall months, which under normal conditions attains its maximum in October and November. If, then, we find it seriously prevalent during the winter months, we must assume the existence of unnatural—and as regards the cases just presented, unnecessary—conditions. So far as known, the reason for its excessive development in cold weather is the one already given. As to the validity of this reason, it may be pointed out that similar winter outbreaks have occurred at Zurich[1] and Geneva, Switzerland. The evidence is therefore now so far multiplied as to render it practically certain that a winter outbreak of typhoid fever means not only sewage contamination of the water supply, but the existence of conditions different from those ordinarily obtaining while the lakes are open. Hence the several outbreaks of winter typhoid, of which we have record in the Great Lake cities and towns are, aside from others, a valid reason why sewage treatment of some sort is rapidly becoming imperative at the Great Lake towns.

AMERICAN SEWAGE-DISPOSAL PLANTS.

MAINE.

Augusta, State Insane Asylum.—So far as known, the credit of the first attempt at sewage irrigation in the United States belongs to Mr. C. B. Laken, treasurer of the State Insane Asylum at Augusta, Maine, who carried out a system of irrigation there about 1872. From a description in the Seventh Annual Report of the State Board of Health of Massachusetts (1876), it is learned that the sewage, which then amounted to about 7,000 gallons per day, first passed by gravity into large tanks, where it was mixed with absorbents—straw, leaves, muck, etc. The solid parts were from time to time carted onto land, while the liquid portion flowed off, to be used for irrigation. In the summer of 1875 an irrigated area of a few acres yielded three crops of fine hay. A portion of the sewage was also used in irrigating a vegetable garden. It is understood that after the death of Mr. Laken this irrigation was abandoned.

NEW HAMPSHIRE.

Concord, State Insane Asylum.—The sewage of this asylum was used for irrigation about the same time as that of the Augusta asylum. In

[1] *For an account of an outbreak of winter typhoid at Zurich, Switzerland, see Mr. Preller's paper on The Zurich water supply, etc.:* Proc. Inst. Civ. Eng., Vol. CXI, pp. 257-295.

this case it was necessary to pump the sewage to the irrigated area. The Eighth Annual Report of the State Board of Health of Massachusetts states that, although the crops were very much increased in value, the sewage had not been disposed of at Concord in the systematic way which would be necessary in dealing with larger quantities. It is not known whether this irrigation is now in use or not.

Rockingham County Poor Farm.—A sewage-disposal plant was first operated at the Rockingham County Poor Farm in 1895. The plant, which is designed to meet the needs of about 1,000 people, consists of two sets of collecting tanks, with filter beds, all so arranged as to admit of intermittent use, the whole inclosed in a building. Sewage is first received in a collecting tank 7 feet wide, 9.5 feet long, and 7 feet deep, where most of the sludge is deposited. The sewage then passes into a second tank, about half the size of the first, and connected therewith at the bottom. The effluent from the second tank flows upon a filter bed of coarse sand, 22 or 23 feet square and 5 feet in depth. The sludge is utilized for compost. The filter areas are ventilated by a force draft over and under them. From the information at hand it is inferred that this plant is a patented process, owned by the Glover Sanitary Sewage Company.[1]

MASSACHUSETTS.

Amherst.—This is a college town in the Connecticut Valley, with a population in 1890 of 4,512. A public water supply was constructed in 1879 and sewerage and sewage-disposal works in 1881. The sewers are on the separate system. In the absence of a water course suitable to receive the sewage, it was conducted to a settling tank, from which the liquid matter is drained onto land through ditches, and the sludge is periodically removed and spread upon land. Ordinarily the sludge is removed once a week by simply opening a valve and allowing it to flow into a bed excavated in the sand, where the water rapidly drains away.

About three-fourths of the sewage of the town is treated in this way, the remainder going to a field having a sandy soil, where it flows into absorption ditches. Three heavy crops of hay are cut yearly from the irrigated area, which previously gave only one light crop a year. This tract is not underdrained, and it is stated that the sewage does not give any trouble by collecting in hollows. The field has a slight slope.[2]

Brockton.—A good recent example of intermittent filtration pure and simple may be seen in the works recently constructed at Brockton, Massachusetts, a shoe-manufacturing town a short distance southwest of Boston, with a population in 1890 of 27,294. Waterworks were completed in 1880 and 1881, but a separate system of sanitary sewers was not put in operation until the latter part of 1894.

---

[1] Refer to Eng. and Bldg. Record, Vol. XXXIII, May 9, 1896.

[2] *Refer to Twenty-fifth* Ann. Rept. Mass. St. Bd. Health; Sewage Disposal in the United States; and *Eng. News, Vol.* XXVIII, July 21, 1892.

A.

B.

BROCKTON, MASSACHUSETTS, FILTRATION FIELDS.

A, Turnips and sweet corn; *B*, Pease, sweet corn and beans.

By the application of meters the use of water at Brockton has been kept quite low. For the year ending November 30, 1894, the average daily consumption per capita was only 24.3 gallons, the highest figure being 39.4, for the month of June, and the lowest 18.7 gallons, for March. In the design of the sewers Mr. Snow, the city engineer, has made allowance for a daily consumption of 60 gallons per capita, as well as for a possible leakage by infiltration into the sewers. The main outlet sewer leading into the receiving reservoir is of brick, egg-shaped, varying from 26 by 39 inches to 22 by 48 inches in size, and about 2 miles long. The leakage by infiltration into the outlet sewer is estimated at about 120,000 gallons per day ordinarily, and in flood time at probably 350,000 gallons.

Inasmuch as there was no area available on which sewage could be delivered by gravity, it was necessary to raise the sewage to the filtration area by pumping. The force main provided for this purpose is of 24-inch cast iron, 16,640 feet in length, and rises 34.5 feet from the pump to the terminal chamber.

The filter area comprises some 30 acres, laid out in 23 beds. The soil of this area is mostly sand for the depth of several feet, but varies considerably in its mechanical constituents. In constructing the beds the natural material was not disturbed except at the surface; on 12 of them the sand was stripped from the subsoil; on 5, a thin layer of subsoil has been allowed to remain, and 6 of the beds have surfaces partly of the top sand and partly of subsoil. The general method of constructing these filter areas is the same as that followed elsewhere, and need not be especially referred to at length. The cost of the disposal works at Brockton is stated at $209,772, of which $59,536 was for the disposal area proper. The main items in the cost of the disposal area are 3,046 square yards of grubbing at $2 per yard, amounting to $6,092; 85,830 cubic yards of excavation in the filter beds, at 23 cents per yard, costing $19,739; trenching for and laying sewage-distributing pipes and underdrains, $12,631; land damages, $9,234. The balance of the $59,536 is made up of miscellaneous small items.

The experience with the sewage beds at Brockton has indicated that furrowed beds are far preferable for winter disposal to flat beds. During the winter of 1894 several of the beds were left with level surfaces, but the surface froze to such an extent on these as to prevent the sewage from penetrating them, whereas beds which had been furrowed the preceding fall took sewage readily during the entire winter. Accordingly, in the winter of 1895–96 all the beds in use were furrowed and no trouble was experienced during that season from freezing. The following, from the reports of the city engineer of Brockton, gives the average amount of sewage disposed of from month to month and its temperature as it reaches the beds.

Average amount and temperature of sewage received at Brockton sewage-disposal works.

Month.	Gallons per day.		Temperature.	
	1895.	1896.	1895.	1896.
			° F.	° F.
January	215,000	474,000	42.6	43.5
February	109,000	568,400	40.6	42.6
March	142,000	648,500	39.5	42.4
April	250,000	475,100	42.3	45.0
May	338,000	321,900	53.5	52.1
June	228,000	381,700	60.1	56.3
July	195,000	322,800	60.8	59.9
August	273,000	365,900	64.0	62.2
September	275,000	527,100	64.0	61.0
October	488,000	600,100	57.0	57.4
November	474,000	616,000	51.0	54.1
December	525,000	599,000	48.0	49.9

According to the report of the Massachusetts State board of health for 1895, the chemical analysis of the sewage and effluent from the underdrains at Brockton, made February 12, 1895, resulted as follows: Of the sewage, the total residue amounted to 84.4 parts per 100,000, of which 40.2 parts were dissolved and 44.2 parts suspended; the total loss on ignition was 38 parts, 12.4 dissolved and 25.6 in suspension; the free ammonia amounted to 3.46 parts per 100,000; the total albuminoid ammonia to 0.91 part, of which 0.244 was dissolved and 0.066 part in suspension; chloride amounted to 9.05 parts; nitrogen, as nitrates and nitrites, nil; the oxygen consumed in an unfiltered sample was 8.56 parts per 100,000, and in the filtered sample 3.12 parts per 100,000; the iron present in an unfiltered sample was 0.32 part, and in a filtered sample 0.08 part per 100,000. The hardness was 6.4.

The effluent of the same date showed an entire absence of turbidity and sediment, while the sewage was thick and contained a heavy black sediment. On Nessler's scale the colors showed 0.01; the residue on evaporation was 4 parts per 100,000; free ammonia, 0.0248 part; albuminoid ammonia, 0.0038; chlorine, 0.75 part; nitrogen as nitrates, 0.045 part; nitrites, nil; the oxygen consumed amounted to 0.024 part per 100,000; hardness, 0.6; and iron entirely absent. In regard to this effluent, it is proper to remark that it is stated to contain a large amount of ground water.

The design at Brockton includes a number of refinements, such as automatic arrangements for gaging the flow of sewage, which are very creditable to the city engineer, Mr. Snow, as well as to the municipal authorities of Brockton in general. Considering the scientific way in which the work there has been started, there is no reason why the

degree of purification to be attained should not be very high, especially since the works are to be operated as intermittent filtration areas only, and we may hope in the future to obtain from Brockton further information of value and interest to all concerned in works of sewage disposal.

During the year 1896, notwithstanding the additional cost of considerable experimental work, the total cost of labor at disposal works was only $2,195.21, or at the rate of $12 per million gallons of sewage treated. This sum included several items, such as setting of trees, grading about building, and pipe testing, which were in the nature of permanent improvements. Labor expended on surface of beds amounted to $932.03; for tending gates, $254.98; and for removing snow, $51.04; a miscellaneous item of $295.40, and a few others making up the remainder. Probably where these works are more fully established the cost of a treatment which yields a very fair effluent will not exceed $7 or $8 per million gallons treated; or, taking into account the value of the crops raised, the works will be about self-sustaining.

In his annual report for 1896 Mr. Snow has given a considerable amount of interesting information as to the operation of the Brockton plant. From this report it is learned that a variety of crops was planted on the beds receiving both sewage and sludge, such as pease, beans, tomatoes, sweet corn, cabbages, and turnips. Some of the conclusions derived from last year's experience are as follows: Pease and beans it is considered will do best if planted on level beds rather than on mounds. The pea crop last year was successful, one of the leading hotels of Brockton taking the entire product. Beans are stated to have failed. Sweet corn, which takes a large amount of sewage and demands but little attention, should be planted 4 feet between rows and in hills 3 feet apart in the rows. Cabbage requires more care than sweet corn, and when sludge is applied to the surface of the bed the same must be irrigated as though the bed were not cropped. It is considered that on a coarse sand or gravel soil with a level surface turnips should receive large doses of sewage, while with heavier soil small doses and a ridged surface will give the best results. Turnips appear to require considerable moisture during the first growth and but little after the root is formed. Six beds were planted with field corn on May 10. The crop was cultivated on June 1, and again about July 1, with a horse hoe. All the beds did well, the yield of one amounting to 144 bushels of ears per acre, or about 70 bushels of shelled corn.[1]

Gardner.—This place is a manufacturing town, situated in the central part of the State, on a divide between the Connecticut and Merrimac rivers. The population in 1890 was 8,424. A sewage-disposal plant by intermittent filtration was constructed in 1891, in conformity with plans approved by the Massachusetts State board of health. This board

[1] Refer to Ann. Repts. of City Eng. of Brockton; Repts. Mass. St. Bd. of Health; and various articles in Eng. News.

required sewage purification at Gardner, because turning sewage into the only available water course would probably create a nuisance. The tract purchased by the town for the disposal area contained about 17 acres of sand and gravelly soil. Much of the material is too fine for efficient filtration. This was overcome by making filter beds largely artificial, they being formed of selected porous sand and gravel from the material. At present there are twelve thoroughly underdrained filter beds, with a total area of 1.04 acres, two supplementary beds not underdrained and with a total area of 0.37 acre, also two other small underdrained beds, containing altogether 0.14 acre. One of the small beds is used to receive sludge from the sludge tank.

The filter beds at Gardner are smaller than those used at Marlboro, Framingham, and other places in Massachusetts, described further on. The sewage is delivered upon the beds by troughs, and each bed is provided with an overflow to prevent washing away the banks in case the sewage should at any time rise too high.

The results obtained at Gardner are said to be satisfactory in summer, but in winter the area is too small for efficient filtration, a difficulty which can be remedied by providing a larger area. The sewage reaches the filter area during the winter at a temperature not much above 40° F.

The cost of preparing beds with piping was about 12 cents per square foot of area. The total cost, including tanks and accessories, was about 14 cents per square foot of filter areas.[1]

Greenfield.—This place had a population in 1890 of 5,252. Sewage has been used for irrigation upon a meadow owned by a private party since 1882. As originally designed, the sewage was systematically distributed over the meadow, but during the last few years, since the death of the former owner, apparently but little effort at such distribution has been made. In consequence, recent statements at hand indicate that the sewage has caused a nuisance.

Lenox.—This place had a population in 1890 of 2,889. A combined system of subsurface and surface irrigation, preceded by tank sedimentation, was adopted in 1876, at which time the population was probably somewhat less than 2,000. The total area originally employed was only about 1½ acres. In October, 1888, a new subsurface-disposal area was placed in operation, which, however, did not cover the whole 17 acres purchased. As to recent results no information is at hand. Lenox enjoys the distinction of being the first town in Massachusetts to construct sewage-disposal works.[2]

Marlboro.—Population in 1890, 13,805. The city is situated on the dividing ridge between the Sudbury and Assabet rivers. The greater part of the drainage, however, naturally flows into Sudbury River,

[1] Refer to Twenty-fourth Ann. Rept. Mass. St. Bd. Health, 1892; Sewage Disposal in the United States, 1894; and Eng. News, Vol. XXIX, Feb. 16, 1893.

[2] Refer to Twenty-fifth Ann. Rept. Mass. St. Bd. Health; Sewage Disposal in the United States; and *Eng. News*, Vol. XXVIII, July 14 and 21, 1892.

from which Boston takes a portion of its water supply. The city of Boston, therefore, had a direct interest in so disposing of the Marlboro sewage as to leave the Sudbury River unpolluted, and, after some negotiations, agreed to pay $62,000 toward constructing the sewage-disposal plant. The sewage flows to the disposal fields by gravity. The total area purchased by the city is 62 acres, upon which forty-four beds for the filtration of sewage and six similar beds for sludge disposal, all covering with their dividing embankments a total area of about 37 acres, have been laid out. Thus far four sludge beds and thirteen sewage beds, or a total of about 11.3 acres, have been prepared for use. The material, which is for the most part a somewhat fine, porous sand with an effective size of 0.12 to 0.14 millimeter, has been thoroughly underdrained by means of parallel lines of 4-inch pipes laid with open joints 50 feet apart and about 6 feet beneath the surface. Filter beds were prepared for use by the removal of the top loam and the leveling of the surface of each at a convenient elevation.

The principal industry at Marlboro is the manufacture of boots and shoes; and, aside from the wastes of the manufacturing establishments, the sewage is ordinary city sewage, varying in strength with the amount of ground water entering the sewers, which were constructed on the separate system. No underdrains were laid beneath the sewers, and during wet seasons a large amount of ground water gains access to them through the joints. The total length of sewers laid to the end of 1895 was a trifle over 22 miles. The amount of sewage is not definitely known. A measurement on May 12, 1892, indicated about 30,000 gallons per day; on May 25, 1892, after heavy rains, the flow was 790,000 gallons per day. No attempt has thus far been made to raise crops on the filter areas.

The results obtained in purification have been satisfactory; over 95 per cent of the organic matter, as represented by the albuminoid ammonia, having been removed. The winter temperature of the sewage as it reaches the fields is from 39° to 42° F. The winter experience indicates that the beds work much better when ridged—as seen in Pl. IV, showing the filter areas at Pawtucket, Rhode Island—than when left level.

It has not been the custom to employ a man constantly in caring for the sewage disposal, but only from time to time as required to change the flow of sewage and to remove scum from the surface of the beds, etc., payment being made by the hour for the time actually consumed. The cost of all work on account of the disposal plant during the year 1893 amounted to $497.19. There seems to be no doubt, therefore, if the amount of labor was sufficient, that the town could recoup itself for the expenditure by preparing more beds and raising crops thereon.[1]

[1] Refer to Twenty-fourth, Twenty-fifth, and Twenty-seventh Ann. Repts. Mass. St. Bd. Health; Sewage Disposal in the United States; and Eng. News, Vol. XXVIII, Aug. 25, 1892.

Concord, Massachusetts Reformatory.—The sewage of this institution was originally discharged into the Assabet River at a point near the institution, but in 1883 an act was passed by the Massachusetts legislature authorizing an expenditure of a sum not exceeding $5,000 for the disposal of the sewage by some other method. The plans adopted include arrangements for pumping the sewage upon an adjacent field, where it is used in broad irrigation of grass and corn crops, as well as other tilled crops to some extent. The soil of this field is light, free, and sandy, with the ground water at a considerable depth below the surface. Portable troughs are used for distributing the sewage to different parts of the field.

The labor is all done by convicts. Assuming a fair valuation for the labor actually used under normal conditions of employment, we have the following as the approximate annual cost of operation: 55 tons soft coal, at $4, $220; salary of attendant, $600; repairs and sundries, $80; total, $900.[1]

Medfield.—Population in 1890, 1,493. Sewers and sewage-disposal works were constructed in 1886, the sewage disposal including preliminary sedimentation and upward filtration through excelsior supplemented by intermittent filtration through natural soil. The material of the filter area is mostly coarse gravel, with ground water about 10 feet below the surfaces of the beds, which are underdrained. There are four beds which, as a general rule, take the sewage in turn for two days. The filter area consists of 1 acre of ground, graded nearly level. The entire cost of the disposal works, including cesspool, pipe from cesspool to outlet, earthwork, engineering, superintendence, and profit to contractor, was about $1,000. The annual expense of maintenance is stated at about $30 a year, the labor being chiefly to change regularly the gate whereby the sewage is diverted from one portion of the filtration area to another.

Analyses of the effluent from the Medfield area, made in October, 1893, showed that the quality was still quite as good as indicated by analyses made when the works were first operated in 1887.[2]

Mystic Valley.—These works, which are operated by chemical precipitation purely, were constructed in 1887 and 1888 by the city of Boston in order to protect the Upper Mystic Lake, whence the water supplies of the Charlestown district of the city of Boston and of the towns of Chelsea, Everett, and Somerville are derived.

Near the head of Upper Mystic Lake, in Winchester and Woburn, there are about a dozen tanneries, the drainage from which formerly passed directly into the lake, which was also considerably polluted by house drainage flowing into the Abbajona River, the influent stream of Mystic Lake. In 1878 a sewer was constructed intercepting all this objectionable drainage and delivering it into Lower Mystic Lake, which

[1] Refer to Seventh Ann. Rept. Mass. St. Bd. of Health, Lunacy, and Charity, Supplement, 1886.

[2] *Refer to Ninetcenth Ann. Rept. Mass. St. Bd. Health; Jour. Assoc. Eng. Socs., Vol. VII, July, 1888. and Eng. and Bldg. Recd., Vol. XVIII, 1888.*

is subject to tidal action. Its delivery there, however, led to controversies between the adjoining towns and the city of Boston, which were finally settled by the construction of the Mystic Valley Chemical Purification Works. Inasmuch as the chief object of these works was to purify manufacturing wastes, they may easily be placed first among successful works of this character. A general view of the works is given in fig. 1 of Water-Supply and Irrigation Paper No. 3.

The works consist in detail of a pump well, connected with the main sewer by a branch sewer of brick, a sewage pump and engine, engine house, four settling tanks, a sludge well, sludge pump, and a series of settling basins for receiving the sludge. In the engine house there are three vats so arranged that the precipitant is fed from the middle vat, which is placed lower than the other two, in which the precipitant is dissolved. These vats are provided with steam coils for heating the water used and with a stirring apparatus driven by the engine. The precipitant is fed to the sewage as it flows from the branch sewer into the pump well, where the process of pumping thoroughly incorporates the chemicals. From the pump the sewage passes into the settling tanks. It is then allowed to remain quiescent for about three hours, when it is found that the sedimentation process is complete. The clarified effluent is then drawn off by removing stop planks. The sludge is pumped, as necessary, into a flume which conveys it to the sludge basins shown in fig. 1 of Water-Supply and Irrigation Paper No. 3.

The cost of the works, including the preparation of the sludge basins, was $10,410.

For the thirteen months ending January 31, 1892, the total quantity of sewage pumped and treated was 133,102,028 gallons. Sulphate of alumina is used as the precipitant, and during this period the total quantity consumed was 331,890 pounds; the amount of coal, 210.7 tons. The quantity of sludge removed during the same period was 2,334 cubic yards, which was mostly carted away by a neighboring farmer for fertilizer. The rate of application of participant was 1 part to 3,354 parts of sewage or 1.24 net tons per 1,000,000 gallons of sewage.

The operation of this plant by the city of Boston ended July 18, 1895, at which time the Metropolitan Sewage Commission assumed control of it. From January 1 to July 18, 1895, the average daily amount of sewage pumped and treated was 376,400 gallons, requiring 143,540 pounds of sulphate of alumina. The Metropolitan Commission has made the Mystic Valley sewer a part of the Metropolitan sewerage system and discontinued the chemical treatment. The first annual report of the Metropolitan water commissioner for the year ending January 31, 1896, states that this plant required the service of about 14 men to operate it, and that its discontinuance removes an annual expense of about $20,000.[1]

[1] Refer to annual reports Boston Waterworks; First Ann. Rept. Met. Water Com.; Sewage Disposal in the United States; a paper by W. F. Larned in Jour. Assoc. Eng. Socs. for June, 1888; and Eng. and Bldg. Recd., Vol. XIX, 1889.

Natick.—Natick is about 17 miles from Boston, on the Boston and Albany Railroad, and had in 1890 a population of 9,118. A public water supply has been in use since 1874. In 1885 Eliot C. Clark, engineer to the Massachusetts drainage commission, recommended that a joint sewage-disposal system be built for the towns of Natick, Framingham, and Ashland, and for the Sherborn Prison, situated a short distance from South Framingham, the principal village in Framingham town, with disposal on a gravelly and sandy area, which has since been selected by Framingham and Natick for this purpose. Mr. Clark's idea was that these several towns and the public institution should act in concert, but, unfortunately, the difficulty of reconciling local differences prevented the adoption of his scheme. The two towns of Framingham and Natick have, however, acted separately in utilizing the area selected by Mr. Clark. Framingham completed its sewerage and sewage-disposal system in 1889, while the Natick system was put in operation only during the year 1896.

The indifference of Natick as to sewage purification has been for a number of years particularly flagrant by reason of the flow of a considerable portion of the town sewage into Pegan Brook, a tributary of Lake Cochituate, one of the main sources of the Boston water supply. Negotiations have been had between the city of Boston and Natick looking to purification of this sewage, but without effect. Finally, a few years ago, the city of Boston constructed, at its own expense, on a sandy plateau adjacent to the lake, a filter area upon which the ordinary flow of Pegan Brook was diverted and purified by intermittent filtration. Some portion of the flood flow of the brook has, however, necessarily gone into the lake whenever heavy rains occurred.

The construction of the Natick works began in 1895, under the direction of Samuel M. Gray as engineer. The detailed plans prepared by him provide for bringing the sewage by gravity to a pumping station through a 30-inch circular brick sewer and 24-inch cast-iron pipe with lead joints. The 24-inch pipe is used where the main sewer passes near the head of Lake Cochituate, in order to prevent any possible leakage into that body of water.

The pumping station is provided with a screen chamber, receiving gallery, storage reservoir, pump wells, etc., and the usual accessories of such a plant. According to a statement made by Mr. Gray, when the sewage reaches the screen chamber it passes through iron bars placed three-quarters of an inch apart, after which it may be turned, as desired, either into reservoirs for storage or into a pump well, from which it may be raised directly to the filter areas. In this latter arrangement the pump can be regulated to take the flow as it comes, thereby saving the lift which would otherwise be necessary if it went to the reservoir. The total capacity of the reservoir is 500,000 gallons. The intention is that the night and Sunday flow be stored in the reservoir.

The sewage is delivered to the filter areas through 8,000 feet of 16-inch cast-iron force main, laid on a grade throughout its entire length, for the purpose of permitting the contents to flow entirely back into the pump well in case it becomes necessary to clean the interior surface of this pipe.

The tract of land appropriated for the disposal works was originally covered with a small growth of wood brush. The soil is sandy and generally well adapted to filtration, although there are thin layers of sandy clay in some of the beds which, as Mr. Gray believes, will make it necessary to lay underdrains. The effluent from the underdrains will be discharged into a small stream which enters the Sudbury River below the outlet of Lake Cochituate. It will be noticed, therefore, that the sewage, as well as the effluent, is entirely diverted from Lake Cochituate, in the same way as in the neighboring town of Framingham.

The entire plant as designed includes the preparation of 12 acres of filters to be underdrained with 3-inch tile, laid on a grade of 2 feet in a hundred, in lines 36 feet apart. The work in 1895 included the preparation of 2 filter beds of 1 acre each, from which the underdrains were omitted. Another bed was completed in 1896 with underdrains.[1]

Norfolk County Jail.—This institution has a land disposal plant of some sort, particulars of which, however, are not at hand.

North Brookfield.—The village of North Brookfield is situated in the town of Spencer, in which a water supply was introduced in 1883. The population of the whole town in 1890 was 8,747, having increased from 7,466 in 1880. North Brookfield proper, however, has a population of only about 3,000, which can be ultimately connected with the main sewer as built in 1893, when sewage-disposal works were also built, the system adopted being that of broad irrigation. According to a statement of Mr. Craig, the supervising engineer of the work, published in the Engineering News of March 22, 1894, it would appear that there are about 20 acres of a farm owned by the town which is well adapted to broad irrigation, the soil being naturally very light and dry, and not yielding heavy crops under ordinary methods of farming. In order to reach the highest point of this area it was necessary to employ an inverted siphon about 1,600 feet in length, which has been successfully constructed and operated, and a profile of which may be seen in Engineering News at the place indicated.

This sewage farm was first in working order in November, 1893, and no statements as to the results of broad irrigation are yet at hand. It is considered, however, that with the favorable soil of the locality very satisfactory results may be attained. This farm is also interesting as being very inexpensive, the total cost of the entire system of sewers, including flush tanks, screen chamber, and labor necessary in preparing the farm, being only $9,700. The sewage is first received in a storage tank, from which it is discharged intermittently to the farm.

---

[1] *Refer to Eng. News*, Vol. XXXV, June 4, 1896.

Pegan Brook.—Some general statements in regard to the filter areas constructed for the purpose of excluding from Lake Cochituate the raw sewage of Natick, which previous to the construction of the Natick sewage-disposal works flowed into Pegan Brook and thence directly into the lake, have been made in discussing Natick (pp. 50–51), and a few detailed statements may now be added. The filter areas are three in number, constructed by removing the soil and placing it in embankments 5 feet high around and between the beds. The filter material, to the depth of 8 feet, is mostly sand, about as fine as ordinary plaster sand. The three beds are about 2, 0.75, and 1.25 acres, respectively. Their elevations are 140, 144, and 146.8 feet above the water-works datum, 134.4 feet being the elevation of low water in the lake. The lowest bed, with an area of about 2 acres, is not underdrained. The water percolates freely through the sand, which is stated to be excellent filter material. Underdrains about 100 feet apart and 8 feet deep are laid beneath the upper beds. They consist of 8-inch vitrified tile pipe, laid with open joints wrapped with canvas. The underdrains were chiefly designed in order that samples of the effluent may be obtained for examination. An earth dam for diverting the water of the brook onto the filter beds and a pumping plant were included in the plans. The entire cost is stated at $12,585.11.

The Pegan Brook filters are of special interest, because, so far as known, they are the first filter areas ever constructed for the filtration of a polluted feeder to a natural lake which is the source of a public water supply. There may possibly be some such arrangement in use abroad, but certainly this is the first plant constructed for such purpose in the United States.[1]

Waltham, School for the Feeble-Minded.—An intermittent subsurface irrigation or filtration plant was constructed at this State institution about 1890. In 1893 permission was granted to connect the sewers of the school with the sewerage system of the city of Waltham, since which time it is understood that the disposal area has been out of service.

Sherborn Prison.—A number of years ago arrangements were made for disposing of the sewage of this State institution by subsurface irrigation. Information as to the extent and success of the project is not at hand. The works are believed to be out of use.

South Framingham.—The sewage-disposal works at South Framingham, Massachusetts, are of considerable interest as involving both broad irrigation and intermittent filtration, and also because real progress has been made here in raising crops on filter beds. Unfortunately, at South Framingham, as well as at about all the other sewage farms in the United States, no one has yet attempted to keep statistics as to the daily flow of sewage, the amount applied, and other important facts, data which must be at hand before definite conclusions can be

[1] *Refer to Ann. Rept.* Boston Waterworks for 1893, and Eng. and Bldg. Record, Vol. XXIX, Apr. 28, 1894.

A.

B.

SOUTH FRAMINGHAM, MASSACHUSETTS, SEWAGE FARM.

A, Sewage inlet to filter (field of growing corn in left foreground, larger corn in distance); *B* Portion of cabbage field.

drawn as to the efficiency of the process at any given point. In favor of South Framingham it may be said that the cropping on the filter beds has been very successful, but as the results have been referred to on page 43 of Water-Supply and Sewage Irrigation Paper No. 3, a further account here is unnecessary.[1]

Wellesley College.—Previous to 1892 the sewage was mostly disposed of by passing it through small and imperfect artificial filters of peat and gravel, but, as the results were unsatisfactory, new works were constructed in that year for treatment by intermittent filtration. Additions to the works were made in 1893. The sewage to be treated is that of about 500 persons, for which purpose a filtration area of 1.55 acres has been provided.[2]

Westboro.—Population in 1890, 5,195. Construction of a sewerage system began in 1891. About 33 acres of land located on the right bank of the Assabet River have been acquired by the town for sewage-disposal purposes, nearly one-half of which may be classed as either swamp or meadow, subject to flooding at times of high water. The sewage is conducted to a settling tank on the disposal area, built in two compartments, with a total capacity of less than 1,000 gallons. Two filter beds, with a total area of a little over 2 acres, were prepared in 1892 by stripping off most of the loam and filling with coarse gravel, the material beneath being a fine compact sand nearly impervious to water. The information at hand indicates that this plant has not worked well.[2]

Worcester.—To show the relative degree of purification obtained by chemical processes of treatment in comparison with intermittent filtration and irrigation processes, brief reference may be made to the results obtained by the works at Worcester, Massachusetts, which may be considered not only the best example of a chemical-purification plant on a large scale thus far completed in the United States, but which, in the opinion of those competent to judge, is as good an example of this method of purification as can be found anywhere. The fact that it has not been successful in entirely mitigating the pollution of the Blackstone River, into which the effluent from the Worcester works flows, must be attributed to inherent difficulties of the chemical process, and not to any lack of proper application of that process at Worcester. Indeed, the author takes pleasure in saying that after having examined the large purification works abroad, as well as a greater portion of those in this country, he has nowhere seen a better example of the chemical treatment, pure and simple, than that at Worcester.

According to the report of the superintendent of sewers for the year 1895, it appears that in 1894, as an average, these works removed 49.35 per cent of the total amount of organic matter present in the sewage as determined by evaporation; of the amount in suspension, 94.45 per

[1] Refer to Report of Com. on Drain. and Sew., etc., Nov., 1889; several articles in Eng. News; and Sewage Disposal in the United States.

[2] Refer to Twenty-fifth Ann. Rept. Mass. St. Bd. Health, 1893.

cent was removed; of the total amount, based on the albuminoid ammonia, 50.79 per cent was removed; of the amount in suspension, based on the albuminoid ammonia, 89.54 per cent was removed; the total amount by oxygen consumed was 43.84 per cent, and the amount in suspension by oxygen consumed 81.08 per cent.

Experience at Worcester indicates that different percentages were removed in the various seasons, as, for instance, in 1894 the total per cent of organic matter removed from the sewage as an average, from July to December, was, by evaporation, 55.02 per cent; the amount in suspension, also by evaporation, 95.76 per cent; the total amount of albuminoid ammonia, 53.78 per cent; the total amount by suspension in albuminoid ammonia, 94.36; total amount by oxygen consumed, 50.23 per cent; the amount in suspension by oxygen consumed, 92.42 per cent.

From December to July the total amount of organic matter removed by evaporation was 42.76 per cent; amount in suspension, 93.08; total amount by albuminoid ammonia, 46.63 per cent; amount in suspension by albuminoid ammonia, 83.26; total amount by oxygen consumed, 37.11; amount in suspension by oxygen consumed, 89.27 per cent.

The average analyses of the sewage for the year ending December 1, 1895, were as follows: Free ammonia, 1.16 parts per 100,000; total albuminoid ammonia, 0.518 part; dissolved albuminoid ammonia, 0.249 part; suspended, 0.270 part. The oxygen consumed in unfiltered sewage was 5.27 parts and in filtered sewage 2.91 parts per 100,000, and the chlorine, 5.98 parts. The average of the effluent for the same year was: Free ammonia, 1.008 parts; total albuminoid ammonia, 0.252 part, of which 0.228 part was dissolved, 0.024 part suspended. In unfiltered effluent the oxygen consumed amounted to 2.4 parts and in filtered effluent 2.27 parts, and the chlorine to 5.92 parts per 100,000. Taking the purification by percentage actually removed, we have, for the free ammonia, 13.02 per cent; total albuminoid ammonia, 51.63 per cent; dissolved albuminoid ammonia, 8.43 per cent; suspended albuminoid ammonia, 91.11 per cent; oxygen consumed, total, 54.35; in solution, 21.99; chlorine, 1.05 per cent. It will be noticed that the chemical process only moderately touches the free ammonia and the dissolved albuminoid ammonia.

The works at Worcester were constructed chiefly because of serious complaints on the Blackstone River as to the pollution caused by the Worcester sewage, which originally flowed into the stream in a raw state. The following gives the mean of a series of analyses of the stream as made by the Massachusetts State board of health in 1895: Color on Nessler's scale, 0.71 part per 100,000; total residue on evaporation, 22.15; loss on ignition, 5.18; free ammonia, 0.3246 part per 100,000; total albuminoid ammonia, 0.0898 part, of which 0.0597 part was dissolved and 0.0301 part was suspended; chlorine, 1.86 parts; nitrogen as nitrates, 0.0267 part; nitrogen as nitrites, 0.0063 part; hardness, 7.3.

A

B

WORKS AT WORCESTER, MASSACHUSETTS.

A, General view of precipitation tanks; *B*, Another view of precipitation tanks.

In considering these results it should be remembered that the sewerage system of the city of Worcester is a combined system, and that frequently at the time of heavy rains a considerable portion of the sewage flows into the Blackstone River entirely without treatment. The information in regard to the Worcester works is too extensive to be given in detail here. It may be found in the several annual reports of the city engineer and the superintendent of sewers of the city of Worcester, and also in the reports of the Massachusetts State board of health.

The litigation between the city of Worcester and the town of Millbury, a few miles down the river, on account of the pollution of the Blackstone River, may, however, be briefly mentioned by reason of its being the most extensive of legal proceedings involving stream pollution thus far taken in the United States. Without detailing all the steps, it may be stated that the objections of Millbury finally resulted in the passage of a mandatory act of the Massachusetts legislature requiring Worcester to purify its sewage before allowing it to flow into the Blackstone River. Under the provisions of this act the purification works were built and first operated in 1890.

In 1890 the population of Worcester was 84,655, while in 1895 it had increased to 98,678, and at the same time the pollution of the river was so bad that in the latter year Millbury again brought suit against Worcester. At the trial, in April, 1896, the case of the town of Millbury was supported by Allen Hazen, Col. George E. Waring, jr., Rudolph Hering, Mrs. Ellen H. Richards, and George W. Rafter, while Worcester's side was represented by Samuel M. Gray, Prof. William T. Sedgwick, Prof. H. P. Kinnicut, Charles A. Allen, Harrison P. Eddy, and F. A. McClure. The testimony touched nearly every phase of sewage purification and stream pollution. In a decree dated December 14, 1896, the presiding justice ordered that within three months Worcester should take steps to purify the stream. The preliminary steps to comply with this decree have been taken.

Worcester Insane Hospital.—These works were constructed in 1876, and have since successfully disposed of the sewage of about 600 people, the ordinary population of the hospital.

As shown on a previous page, some attempt has been made to dispose of sewage by broad irrigation at the State insane hospitals at Augusta, Maine, and Concord, New Hampshire, a short time before the sewage farm of the Worcester Insane Hospital was first operated. Neither, however, was entirely successful, and both have been abandoned. We must therefore give the credit to the authorities of the Worcester Hospital for instituting the first permanently successful sewage-irrigation farm in the United States.

The hospital building is situated on a considerable rise of ground, about 3,000 feet west of the main irrigation field, which includes 14 acres. There are also arrangements for drawing sewage along the line of the sewer leading from the hospital to the main field, which permits

the irrigation of a total of from 30 to 40 acres. The entire hospital farm includes 257 acres.

The work was mostly constructed by the inmates of the hospital and no statement of cost can be made.[1]

PLANTS PROJECTED IN MASSACHUSETTS.

Information as to the number of projected sewage-purification plants in Massachusetts has been obtained from the annual reports of the State board of health, 1890 to 1895, inclusive, as follows:

Andover.—This town submitted plans for sewage disposal to the State board of health for approval in 1895. The plans proposed were rejected as thoroughly impracticable, and the sewer commissioners were instructed in a communication dated October 10, 1895, as to the proper lines on which to develop sewage purification for that place.

Braintree.—The sewerage committee of this town submitted plans for sewage disposal to the State board of health in 1893. In its answer, under date of July 6, 1893, the board suggested that inasmuch as the towns of Braintree and Weymouth were contiguous and were near the sea, the interests of both communities might be served by united action in the construction of an outlet to tide water.

Concord.—The committee on sewage of this town asked the advice of the State board of health on a proposed plan of sewage disposal in 1894. Under date of January 3, 1895, the board replied that, judging from the outline presented, the plan proposed could be commended as well adapted to the needs of the town, and that a system of intermittent filtration on the area proposed would be a proper method of disposing of the sewage of Concord.

Danvers.—In 1895 the board of health of this town applied to the State board for advice as to the disposal of sewage of three morocco factories in that place. In its answer, under date of December 5, 1895, the State board of health suggested, as a line of investigation, that probably the sewage should be first conducted into large settling tanks, where the sludge would deposit, and from which the liquid could be pumped to a filter bed and the sludge be frequently removed from the tanks, to be disposed of presumably either by burying or for use as manure in agriculture.

Foxboro Hospital for Dipsomaniacs and Inebriates.—Information as to an intermittent filtration plant proposed for this State institution may be found in the Twenty-third and Twenty-fourth Annual Reports of the Massachusetts State Board of Health.

Leicester.—Plans for sewage disposal for this town were submitted to the State board of health in June, 1893. They included preliminary sedimentation and filtration, the sludge to be allowed to drain out in beds, after which it will be carted away for use in agriculture.

[1] Refer to the Forty-seventh Ann. Rept. of the Trustees of the Hospital for the year ending September 30, 1879.

Medfield Insane Asylum.—Plans for the disposal of the sewage of this State institution by means of intermittent filtration were submitted to the State board of health in 1892.

Pittsfield.—Chapter 357 of the acts of 1890 authorizes the city of Pittsfield to build a system of sewerage and sewage disposal, subject to the approval of the State board of health. The plans were presented to the board for its approval in 1891. They provide for the permanent disposal of the sewage by intermittent filtration, but permit a temporary discharge into the Housatonic River until June 1, 1900, after which date the sewage of the city must be treated before discharge, as provided in the plans.

Spencer.—This town was advised by the State board of health, under date of December 8, 1893, as to proper disposal of the town sewage by land treatment. The board pointed out that the wastes from the gas works which now enter the sewers of Spencer should be either excluded or so treated before admission as to remove all tarry or oily substances which would tend to clog the filters.

Bridgewater, State farm.—Under date of December 14, 1895, the State board of health advised the superintendent of the State farm at Bridgewater as to the purification of the sewage by land treatment, preferably by filtration through porous soil about 1¼ miles from the building.

Tewksbury.—The State board of health advised the authorities of this town in regard to land purification of the sewage under date of October 3, 1895.

Wakefield.—In 1892 the sewerage committee of Wakefield submitted to the State board of health plans for the sewerage and sewage disposal of the town, the sewage of the principal village of the town to be treated by filtration upon land, and the remainder of the sewage to be discharged into the main metropolitan sewer. In its answer to the application, under date of October 7, 1892, the State board of health gives general suggestions as to the best method of preparing the disposal areas.

At several other places in Massachusetts sewage-disposal works have been projected and are likely to be constructed in the next few years. The foregoing examples are sufficient to indicate the advanced state of sewage-purification questions in the Commonwealth of Massachusetts.

RHODE ISLAND.

Sewage-disposal works have been either constructed or projected in Rhode Island as follows:

Central Falls.—The present city of Central Falls was part of the old town of Lincoln, in Providence County. The population of the town of Lincoln for 1890 is given at 20,355. In 1892 a tract of land of about 35 acres was purchased for the disposal of the sewage of the western district of the city, at a cost of $20,000. The disposal plant, which for the present includes three filter beds of about one-fifth of an acre each,

was constructed in 1896. Additional filter areas are to be constructed as required. The sewage flows to the disposal grounds by gravity. The cost of the work of preparation to date, including settling tank, is is about $9,500.[1]

Pawtucket.—Pawtucket, with a population of 32,577, according to the State census of 1895, is divided into two principal drainage districts, known as the Blackstone River district and the Moshassuck River district. The sewers of both the districts are of the combined system, those of the first district amounting to 27.16 miles, and those of the second district to 7 miles. The filter plant is connected with the sewers of the second district, which includes an area within the city of 960 acres and an approximate population of 9,500; 110 acres of this

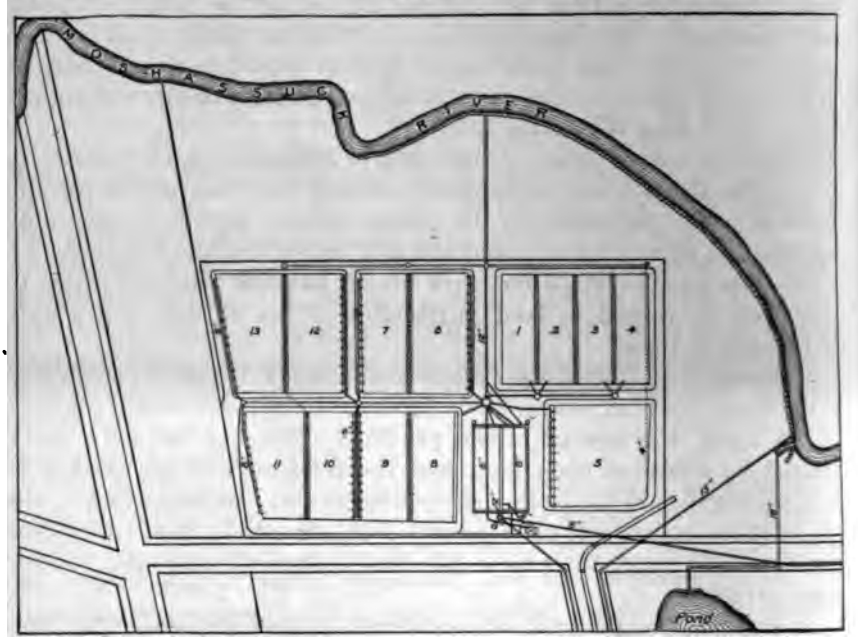


FIG. 4.—Plan of filter areas at Pawtucket, Rhode Island.

area is the most thickly settled portion of the city, having a stated population of 34 persons to the acre. Filter areas are located in the southern part of the city and border upon the Moshassuck River, as shown in Pl. IV. This stream is comparatively small, having an ordinary flow of from 30 to 45 cubic feet per second. It is very much polluted, both above and below the filter areas, by wastes from manufactories, and in view of future complications it was deemed best that the city should not be a party to this pollution. The filter areas, as already constructed, include a little less than 4 acres, about 2.4 acres being the actual area capable of receiving sewage. This is divided, as shown in fig. 4, into 13 beds, 4 of which are designed for sludge beds,

[1] Refer to Eng. and Bldg. Recd., Vol. XXXV, April 24, 1897.

FILTER AREAS AT PAWTUCKET, RHODE ISLAND, AS RIDGED FOR WINTER SERVICE.

[redacted]

and are laid out slightly lower than the other beds in order to receive the sludge from the bottom of the tanks.

Covered sewage tanks have been provided, into which the sewage is first received, and from which it is discharged upon the beds as desired. The sewage flows to these tanks from the city by gravity. Arrangements have also been made by which the flow of sewage can be shut off from the tanks automatically and turned into the storm overflow whenever the amount delivered is so increased by a storm as to tax the capacity of the tanks above the average or above any amount to which the apparatus has been adjusted. After the cessation of the storm and the consequent return of the sewage to the average rates, the gates to the settling tank open automatically, allowing the sewage to again flow into them. In ordinary use sewage is received into the tanks for about twenty-four hours and then turned upon the beds under irrigation at the rate of 100,000 gallons per acre.

Work was begun on the disposal areas in 1893. Generally speaking, the work at Pawtucket has been well carried out and the operation of the plant is successful.[1]

Cranston, State institutions.—The State institutions at Cranston, Rhode Island, include a house of correction, State almshouse, State hospital for the insane, State prison, the Sockanosset School for Boys, and the Oak Lawn School for Girls. They are situated in the midst of a tract of about 500 acres of land owned by the State. Sewage disposal by land treatment was first proposed at Cranston in 1884, and carried out in 1895, since which time it has been successfully operated.

The filtration is said to work well in winter, the sewage reaching the field ordinarily at temperatures of about 40°, 42°, or 43° F.[2]

Providence.—Sewage disposal on a large scale has also been projected for the city of Providence, which had a population in 1890 of 132,146. As the matter now stands, it is proposed to partially purify the sewage by chemical precipitation and discharge it into tide water at Fields Point. Main intercepting sewers have been under construction for the last few years with this end in view.[3]

Woonsocket.—Sewage-disposal works have also been projected for Woonsocket.

CONNECTICUT.

Bristol.—A recent example of a combination broad-irrigation and intermittent-filtration plant is at Bristol, Conn. The population of this town in 1890 was 7,382. In 1892 a number of suits were brought to restrain the discharge of crude sewage into a stream flowing through

[1] Refer to Eng. News, Vol. XXXV, January 2, 1896, where a very complete account by George A. Carpenter, city engineer of Pawtucket, designing and constructing engineer of these works, may be found.

[2] Refer to Seventeenth Ann. Rept. Bd. Charities and Corrections of R. I., 1895; also Eng. and Bldg. Record, Vol. XIII, March 4, 1896.

[3] The Providence sewage-disposal data are very extensive; for a digest of the same refer to Sewage Disposal in the United States.

the town. The suits have been held in abeyance pending the construction of sewage-purification works.

Plans for a sewerage and sewage-disposal system were prepared in 1892, but the actual work of construction was delayed about three years awaiting necessary legislation. Acting under the advice of Mr. McKenzie, the town's engineer, 80 acres of land were purchased at $35 per acre for the purpose of constructing filter beds and also for utilizing sewage by broad irrigation. The necessary sewers and a portion of the disposal area were constructed in the year 1895.

The filter areas as thus for prepared include about 6 acres. The material is coarse sand and gravel. Sewage is distributed to them by means of 15-inch and 12-inch carriers. The underdrains, so far as laid, are 8 inches and 6 inches in diameter and 25 feet apart. It is not intended to underdrain the irrigation area. The cost of preparing 6 acres of filter beds, with the necessary appurtenances, was about $9,000. No definite statements can yet be made as to the results of purification or irrigation at this place.

Danbury.—Population in 1890, 16,552. A few years ago the Wolf disinfecting process, which consists in dosing the sewage with sodium hypochlorite formed by a partial electrical decomposition of a solution of salt and water, was applied. The process, as applied at Danbury, proved inefficient, and suits were brought enjoining the city from discharging its sewage into Still River. The court recommended intermittent filtration, adjudged costs against the city, and required discontinuance of the discharge after May 1, 1897. Under these circumstances the town purchased an extensive tract of land, a portion of which is stated to have been prepared for use in intermittent filtration.[1]

Lake Wauremaug.—This lake, which is surrounded by a number of hotels and cottages, has abrupt, rough, and rugged shores, which render an economical, safe disposal of sewage somewhat difficult. Areas for land treatment are entirely lacking. Under these circumstances a committee of the State board of health in 1894 recommended the treatment of all human fæces and urine by a dry-earth system, and the treatment of laundry and sink wastes by filtration in trenches on sloping ground, somewhat after the method successfully used at Wayne, Pennsylvania. It is understood that these recommendations have since been carried out.

Litchfield.—Population in 1890, 3,304. The first sewage-irrigation field in Connecticut was inaugurated at this place in 1890. Area of same, 4 acres. No details as to method of operation or success attained are at hand.[2]

Manchester.—Population in 1890, 8,222. It is stated that a sewage-disposal plant by land purification has recently been constructed. No information as to details is at hand.

---

[1] Refer to Eng. News, Vol. XXXIV, August 29, 1895.

[2] Refer to Fourteenth Ann. Rept. Conn. St. Bd. Health, 1891.

Meriden.—The sewage-disposal works at Meriden furnish an example of combined irrigation and intermittent filtration. The population of Meriden was 21,652 in 1890. Waterworks were built in 1869.

The gradual extension of sewers having led to a considerable pollution of Harbor Brook, a small stream flowing through the central and lower part of the town, the question of sewage purification gradually became a leading one until a special committee was appointed, in 1891, to investigate and report on the general question. This committee submitted reports of engineers recommending broad irrigation, supplemented by intermittent filtration, on a tract of land of about 150 acres, which was subsequently purchased for this purpose. According to statements in Engineering News, the disposal area as originally planned included some 32 acres for broad irrigation and 14 filter beds, 8 of which are 150 feet by 350 feet and the balance one-half that size. That portion of the disposal area which has been laid out in beds is stated to consist of fine soil 3 feet deep, below which is a layer of sand and gravel of unknown depth, admirably suited for filtration. The fine, deep soil contains a layer of rather poor, brownish-black material about 8 inches thick, below which is red loam sand, finely divided and nearly without grit. The sand and gravel farther down are very coarse, most of the sand being coarser than ordinary building sand.

In constructing the filter areas only about 12 inches of the top soil was removed, leaving about 2 feet of the fine material above the coarser subsoil. The filter beds have been underdrained with 6-inch tile placed 2½ feet below the surface at their upper end and 5 feet below at their lower ends. Inspecting chambers are placed at the junction of each under drain with the main drain. The main drain discharges into water carriers leading to a neighboring river.

Sewage was first discharged upon the original filter areas in September, 1893. It was then reported that very little effluent appeared in the underdrains, and that the discharge from them grew less and less until finally practically none appeared. The clear inference is that the effluent passed on down below the drain pipes and sank into the coarse material below.

Considerable trouble has ensued at Meriden, chiefly by reason, apparently, of neglect of the fundamental principles of sewage purification and utilization, not only as worked out in detail by the experiments of the Massachusetts State board of health, but as practiced successfully for many years abroad. The beds have not only been overdosed, but in their original preparation the exceedingly fine material above the subsoil was partially left instead of being fully removed, with the result that the filtration was not only very slow, but the filtering material was soon clogged. In 1895 all the beds were stripped of top soil and fine sand. Since that time the results have been much better.

So far as can be learned, no special attempt has yet been made to

grow crops upon the filter area, although the land utilized was selected by the designing engineer, Mr. Bassett, with special reference to such use. Possibly future reports from Meriden may indicate more satisfactory results. An attempt at sewage farming on a large scale at this disposal area would be watched with great interest by everybody. Markets for all kinds of produce are near at hand, and there seems to be no reason why, with proper management, a sewage farm here might not be very successful.[1]

Waterbury.—This town had a population in 1890 of 28,646. As we have seen on a previous page, the pollution of Naugatuck River by manufacturing wastes is very serious. It is understood that plans are now under way looking toward the purification of the sewage of Waterbury by land treatment.

NEW YORK.

Brewsters.—The so-called system of electrical treatment as used at Danbury, Connecticut, was also applied by the New York City department of public works for treating both sewage and water at Brewsters in the summer of 1893. So far as can be learned, the process has not been any more successful at Brewsters than at Danbury. In a paper read by Dr. Thomas M. Drown before the New England Water Works Association on March 14, 1894, it is shown that the so-called electrical purification is simply a process of disinfection by sodium chloride, in which electricity has nothing further to do with the process than that sodium hypochlorite is produced from salts naturally present in the sewage by electrolysis. It is probable that the cost of any treatment which includes the preparation of sodium hypochlorite by electrolysis is more expensive than it would be to buy this salt in the market and apply it by the ordinary methods of mixing used in chemical purification.

Brooklyn, Twenty-sixth ward.—Chemical-purification works have been recently constructed in the Twenty-sixth ward of the city of Brooklyn, in general accordance with the patented process of J. J. Powers. This process has also been carried out at Coney Island, Echota, Round Lake, White Plains, Sheepshead Bay, and other places in the State of New York. As originally operated, perchloride of iron was the only precipitant used, but in the more recent plans lime is also added and the sludge is further dosed with chlorine gas. Perchloride of iron costs about 3.5 cents a pound in New York. Chlorine is prepared from common salt, black oxide of manganese, and sulphuric acid. As to the cost of operating this system, the following approximate estimate of the daily expense at White Plains may be given:

[1] The works at Meriden were reported upon by Messrs. T. H. McKenzie, Simpson C. Heald, and Carroll P. Bassett. As to the views of each of these engineers, etc., see Eng. News, Vol. XXXIV, July 18, 1895, where an extended account of the works may be found.

Estimate of daily expense at chemical-purification works at White Plains, New York.

1 carboy of perchloride of iron	$4.75
1 barrel of lime	.75
Coal	.90
Engineer	2.25
Laborer and watchman, each, $1.50	3.00
Common salt, black oxide of manganese and sulphuric acid	.50
Oil and waste	.30
Miscellaneous	.50
Total	12.95

At the time this estimate was made, the average daily quantity of sewage treated was about 250,000 gallons. Assuming an average cost for treating this quantity of $12 instead of $12.95, we reach the figure of $48 per million gallons treated, which figure may, however, be expected to decrease somewhat with larger quantities of sewage treated. The figures show, however, that this is an exceedingly expensive process under any circumstances, as will be more readily realized when it is stated that the land processes, aside from yielding returns from the crops, do not ordinarily cost more than a few dollars per million gallons treated.

Chautauqua Assembly.—Chemical-precipitation works were constructed at the Chautauqua Assembly in 1893. This place has a summer population at times of several thousand. Measurements of the flow of sewage July 24, 1893, showed a total of 122,884 gallons in twenty-four hours.

According to an advertisement issued by the committee in charge of the Assembly grounds in October, 1892, the plans were required to embody a complete plant capable of sending out a clear and odorless effluent harmless to fish life, the object of the works being chiefly the maintenance of the cleanliness of the Chautauqua Lake beach in front of the Assembly grounds. The plans adopted were those of W. B. Landreth, and include the use of lime and sulphate of alumina as precipitants. The following analysis, made by Prof. Maurice Perkins, of Union College, in September, 1893, indicates the degree of purification obtained at this plant.

Analyses of water from chemical-precipitation works at Chautauqua Assembly.[1]

[Parts per 100,000.]

	Sewage.	Effluent.	Per cent removed.
Total solid residue	117.0	23.6	79.8
Mineral matter	59.2	14.4	75.7
Loss on ignition	57.8	9.2	84.1
Chlorine	6.0	6.0	0.0
Free ammonia	24.28	1.357	44.1
Albuminoid ammonia	2.088	.486	76.9

[1] Mr. Landreth has recently stated to the author that the results of this analysis have been fully verified by repeated analyses made each season since 1893.

If the results given in the foregoing are maintained permanently the Chautauqua Assembly sewage-disposal plant may be regarded as of a very high order.

In 1893, from July 1 to October 1 (ninety-two days), the cost of labor was: Superintendent and engineer, $255; helper, $131; total, $386, or at the rate of about $4.20 a day. The amount of chemicals used per diem is stated to average about as follows: Lime, 1 barrel; alumina sulphate, 30 to 40 pounds; copperas, 2 to 5 pounds. Fuel is estimated to cost about 30 cents a day. Taking labor at $4.20, fuel at 30 cents, and chemicals at $1.75, we reach a total of $6.25 a day. Assuming an average attendance of 4,000 people during the season, the daily cost per capita is found to be 0.156 cent, or at the rate of 57 cents per capita per year. Chemicals on the foregoing basis cost about 16.1 cents per capita per annum. Taking the actual flow of sewage as averaging about 125,000 gallons a day, we reach a cost per million gallons treated of $8.88 for lime, $7.76 for alum, and $1.28 for copperas, or a total of $17.92 per million gallons treated, which again indicates somewhat greater expense for chemical treatment than would be required for treating the same amount of sewage on land. Inasmuch, however, as the results were largely experimental in 1893, it is possible that later figures would show a marked reduction in cost. Chemical treatment will, however, even under the most favorable circumstances, cost more than land treatment, other conditions being, of course, assumed as equal.[1]

Far Rockaway.—Population in 1890, 2,288. Plans for sewerage and sewage disposal were approved by the New York State board of health in 1895; they include the collection of the sewage of the village at central points, from which it is to be pumped, by means of an electric motor and motor pumps, to a disposal field, where it will be purified by land treatment. It is understood that the work is at present under construction.

New Rochelle.—Population in 1890, 8,217; at present it is perhaps 11,000. A chemical purification plant, designed to treat eventually [illegible] gallons of sewage a day, was carried out in 1894 at an estimated cost of about $19,000. In addition to this sum, 2 acres of land purchased, right of way, legal expenses, etc., were estimated to cost about $4,380, or a total of about $23,380. Statements are lacking as to either the efficiency of the process or just the method of treatment pursued.[2]

Hemlock Lake.—The domestic water supply of the city of Rochester is obtained from Hemlock Lake, about 30 miles southerly from and at an elevation of [illegible] feet above the city. The works for bringing this supply to Rochester were constructed in 1874 and 1875, about which time the attention of the citizens of Rochester as well as those of the

[1] Note to Eng. News, Vol. XXXI, February [illegible], [illegible].
[2] Note to Eng. News, Vol. XXXIII, February [illegible], [illegible], and Eng. and Bldg. Record, Vol. XXXI, [illegible].

surrounding cities and towns was attracted to the lake as a desirable point for summer residence. In 1892 there were in use about 120 cottages and several hotels and summer boarding houses, and the summer population about the margin of the lake, including transient visitors, was from 800 to 1,000 persons.

In order to care for the sewage and house wastes the city of Rochester constructed in 1886 a pail system by which the night soil and garbage from the cottages and hotels are collected and removed to the foot of the lake, where the material is deposited in narrow trenches in thin layers, and, after treating with lime, each layer is covered with about 6 inches of dry earth. This process is repeated daily until the trench is nearly filled, when the remainder is rounded up with earth and a new trench is started. The surface of the full trench is then cultivated and cropped, and after a suitable period the same land is again used. The trenches were originally made about 3 feet deep, but experience indicates much more rapid destruction of the organic matter near the surface than at the lower levels. It has been found that after a period of three years the same trench can be used again. The plant includes a building with convenient arrangements for washing the pails during the summer and for storing them in winter, as well as a flat-bottomed scow for transporting the sewage receptacles on the lake, and a tramway and pier, with a total length of 2,200 feet, by which they are transported from the lake to the place of treatment. The approximate cost of the plant to April 6, 1891, including surveys, building, scow, wagon road, tramway, carts, pails, and land, was about $10,000. The yearly cost of operation is from $1,500 to $1,700. In winter only a few of the dwellings are in use and the collections are made weekly by wagon.[1]

Bath, Soldiers' and Sailors' Home.—Mr. Power's patented process, described on page 62, is in use at this place, where the permanent population may be taken at from 1,200 to 1,500. Chemicals are stated to cost fully $2,000 a year. Including fuel, labor, and other expenses, the sewage purification costs about $3 per capita per annum. A good opportunity exists at this State institution to elevate the sewage by pumping onto porous lands owned by the State just back of the home, where it could be efficiently utilized in irrigation.

Batavia, State Institution for the Blind.—The average quantity of sewage of this institution is estimated at about 10,000 gallons a day, this quantity being based upon an average daily consumption of water of 50 gallons per capita for the 200 officers and inmates. Chemical-treatment works costing about $6,500 and an outlet sewer costing $6,000, or a total of $12,500, were constructed in 1890–91. No statements as to the efficiency of the process are at hand.

Vassar College.—Vassar College is situated near the town of Pough-

[1] Refer to Ann. Repts. Ex. Bd. City of Rochester, and Sewage Disposal in the United States. The city of Rochester is now (1899) purchasing a strip of land about Hemlock Lake, including all the cottages, and this collection system will accordingly soon be discontinued.

keepsie, a short distance back of the city, upon a small stream, which reaches the Hudson River by a roundabout course of something like 6 miles. For a number of years previous to 1895 the college sewage flowed to a series of covered tanks, where it underwent a rapid filtration through peat, gravel, and other substances. The effluent from this process, while considerably clarified, was still objected to as unfit to pass into a small outlet creek, the township health officer alleging that its discharge was injurious to riparian owners below.

After considerable discussion, in which various projects were suggested, a plan for disposal by intermittent filtration and broad irrigation was prepared and carried out by Messrs. Noyes and Hazen in the fall of 1895. Under this plan the sewage first passed into a screening chamber provided with a screen rack formed of iron bars ¼ by 2 inches in diameter, placed seven-eighths of an inch apart. The sewage enters the screening chamber parallel to the screen rack and not against it. The matter collected at the screen is taken out each day, and after draining on a platform is removed to the boiler house and burned. From the screening chamber the sewage flows to a reservoir of 44,000 gallons capacity, from which it was pumped to the disposal area, which consists of two filter beds with a total area of 1.03 acres exclusive of the embankment. The filtering material is a rather coarse sand with an effective size of grain of from 0.30 to 0.40 millimeter.

The level of the ground water was found to be about 15 feet below the prepared surface of the filter areas, and as there was a gully near by which afforded a free outlet for the ground water, underdraining was deemed unnecessary.

The filter beds provided are regarded as sufficient to receive the entire sewage of the college, but as there was a considerable area of sandy soil near by at a somewhat greater elevation, the force main was extended to a point commanding this area, so that during the growing season several acres of land can be utilized for broad irrigation. In this way the fundamental principle of effective sewage utilization has been recognized in these works, the sewage going to the land when necessary for the best results of agriculture and to the filter areas when not needed on crops, thus insuring an efficient filtration at all times without reference to cropping.

Complete records as to the quantity of sewage do not appear to have been kept, although during the months of February and March, 1896, the average daily quantity is stated as 56,000 gallons, the range being from 32,000 to 113,000 gallons. The sewers receive roof water and the higher quantities are stated as due to accessions from this source.

The total cost of the works as constructed, including the preliminary expense involved in considering the various projects proposed before the final adoption of the sewage-utilization project, is stated at $7,500, *exclusive of the* land.

Chemical examinations have been made of the water taken from the brook receiving the effluent, above the point where the effluent enters as well as below, with the result of showing that the stream below the effluent now carries no trace of sewage contamination. A great improvement in the condition of the stream is also shown by analyses made before and since the beginning of the filtration by Mrs. Ellen H. Richards, one of the trustees at Vassar College, and analyst of the Massachusetts State board of health. The examinations are to be continued for considerable length of time in order to gain information as to the final effect on the effluent and as to other questions of interest to those planning similar works. The enterprise of Vassar College in carrying out so thorough a system and also in causing systematic examinations to be made for the purpose of acquiring information for future guidance is highly commendable.[1]

PLANTS PROJECTED IN NEW YORK.

Albion.—Population in 1890, 4,586. Plans for sewage disposal by intermittent filtration were approved by the State board of health in 1890. The sewage will be delivered at the filtration fields by gravity, and the topographical conditions are such as to admit of utilization in agriculture by broad irrigation on porous, sandy lands.

Cobleskill.—Population in 1890, 1,822. Plans for sewage disposal by chemical purification were approved by the State board of health in 1893. No information is at hand as to whether or not land-purification processes have been considered.

Cortland.—Population in 1890, 8,590. Plans for purification of the sewage of the town by chemical treatment were approved by the State board of health in October, 1893. The plans also include possible filtration through land, should a high degree of purity be found necessary. It is proposed to use lime and sulphate of alumina as precipitants. The plans, as approved, include a tank area for from 9,000 to 10,000 people, with arrangements whereby the capacity can be increased as required. The report of the engineer does not state whether or not the locality would admit of land-purification processes.

Flatbush.—Under the health laws of the State of New York, the governor may, on presentation of evidence as to the existence of a nuisance, order the same to be abated under plans subject to the approval of the State board of health. In 1894 Governor Flower issued such an order with reference to the sewage of the village of Flatbush, on Long Island, which then grossly polluted Mill Creek. No information is at hand as to the action finally taken under this order.

Fort Plain.—Population in 1890, 2,864. This town is situated in the narrow portion of the Mohawk Valley at a place where little opportunity for land purification exists, except that the town incurs the expense of pumping. The cities of Rome, Little Falls, and Utica, on the Mohawk River above Fort Plain, discharge sewage into the river without treat-

[1] For an account of the Vassar College works, see the Engineering Record of June 21, 1890.

keepsie, a short distance back of the city, upon a small stream, which reaches the Hudson River by a roundabout course of something like 6 miles. For a number of years previous to 1895 the college sewage flowed to a series of covered tanks, where it underwent a rapid filtration through peat, gravel, and other substances. The effluent from this process, while considerably clarified, was still objected to as unfit to pass into a small outlet creek, the township health officer alleging that its discharge was injurious to riparian owners below.

After considerable discussion, in which various projects were suggested, a plan for disposal by intermittent filtration and broad irrigation was prepared and carried out by Messrs. Noyes and Hazen in the fall of 1895. Under this plan the sewage first passed into a screening chamber provided with a screen rack formed of iron bars ¼ by 2 inches in diameter, placed seven-eighths of an inch apart. The sewage enters the screening chamber parallel to the screen rack and not against it. The matter collected at the screen is taken out each day, and after draining on a platform is removed to the boiler house and burned. From the screening chamber the sewage flows to a reservoir of 44,000 gallons capacity, from which it was pumped to the disposal area, which consists of two filter beds with a total area of 1.03 acres exclusive of the embankment. The filtering material is a rather coarse sand with an effective size of grain of from 0.30 to 0.40 millimeter.

The level of the ground water was found to be about 15 feet below the prepared surface of the filter areas, and as there was a gully near by which afforded a free outlet for the ground water, underdraining was deemed unnecessary.

The filter beds provided are regarded as sufficient to receive the entire sewage of the college, but as there was a considerable area of sandy soil near by at a somewhat greater elevation, the force main was extended to a point commanding this area, so that during the growing season several acres of land can be utilized for broad irrigation. In this way the fundamental principle of effective sewage utilization has been recognized in these works, the sewage going to the land when necessary for the best results of agriculture and to the filter areas when not needed on crops, thus insuring an efficient filtration at all times without reference to cropping.

Complete records as to the quantity of sewage do not appear to have been kept, although during the months of February and March, 1896, the average daily quantity is stated as 56,000 gallons, the range being from 32,000 to 113,000 gallons. The sewers receive roof water and the higher quantities are stated as due to accessions from this source.

The total cost of the works as constructed, including the preliminary expense involved in considering the various projects proposed before the final adoption of the sewage-utilization project, is stated at $7,500, exclusive of the land.

ıemical examinations have been made of the water taken from brook receiving the effluent, above the point where the effluent rs as well as below, with the result of showing that the stream w the effluent now carries no trace of sewage contamination. A t improvement in the condition of the stream is also shown by yses made before and since the beginning of the filtration by Mrs. n H. Richards, one of the trustees at Vassar College, and analyst he Massachusetts State board of health. The examinations are e continued for considerable length of time in order to gain infor- on as to the final effect on the effluent and as to other questions of rest to those planning similar works. The enterprise of Vassar ege in carrying out so thorough a system and also in causing sys- ıtic examinations to be made for the purpose of acquiring informa- for future guidance is highly commendable.[1]

PLANTS PROJECTED IN NEW YORK.

lbion.—Population in 1890, 4,586. Plans for sewage disposal by rmittent filtration were approved by the State board of health in . The sewage will be delivered at the filtration fields by gravity, the topographical conditions are such as to admit of utilization in culture by broad irrigation on porous, sandy lands.

bleskill.—Population in 1890, 1,822. Plans for sewage disposal by ıical purification were approved by the State board of health in . No information is at hand as to whether or not land-purification esses have been considered.

rtland.—Population in 1890, 8,590. Plans for purification of the ıge of the town by chemical treatment were approved by the State d of health in October, 1893. The plans also include possible filtra- through land, should a high degree of purity be found necessary. proposed to use lime and sulphate of alumina as precipitants. The s, as approved, include a tank area for from 9,000 to 10,000 people, arrangements whereby the capacity can be increased as required. report of the engineer does not state whether or not the locality ld admit of land-purification processes.

atbush.—Under the health laws of the State of New York, the gov- r may, on presentation of evidence as to the existence of a nuisance, r the same to be abated under plans subject to the approval of the e board of health. In 1894 Governor Flower issued such an order reference to the sewage of the village of Flatbush, on Long Island, h then grossly polluted Mill Creek. No information is at hand as ıe action finally taken under this order.

rt Plain.—Population in 1890, 2,864. This town is situated in the ow portion of the Mohawk Valley at a place where little opportu- for land purification exists, except that the town incurs the expense ımping. The cities of Rome, Little Falls, and Utica, on the Mohawk r above Fort Plain, discharge sewage into the river without treat-

[1] For an account of the Vassar College works, see the Engineering Record of June 27, 1896.

at the present time it is doubtful if any improvement is effected. The answer to a recent letter of inquiry refers to the Atlantic City sewage-purification works as a great farce.

East Orange.—The population of this place in 1890 was 13,282. Sewage-disposal works with chemical treatment supplemented by filtration through land were constructed in 1887 and 1888. The chemicals used are lime and sulphate of alumina. So far as known, no attempt to raise crops on the filter areas has ever been made. The works are now out of use, the sewage going to the Passaic River by way of the Newark sewers.

Essex Fells.—This place is an improved suburban-residence district, with a daily flow of sewage of about 20,000 gallons. Originally the purification was merely such as could be obtained by sedimentation in a tank, supplemented by the flow of the effluent through a mass of broken stone, and thence into gravel. Two filter areas capable of purifying double the present flow were constructed early in 1897.[1]

Freehold.—Population in 1890, 2,932. A disposal plant by broad irrigation was brought into use in 1893. For this purpose 16 acres of land were purchased, upon which the sewage is delivered by gravity. A portion of this has an unfavorable peaty soil with a surface of from 6 to 12 inches of bog iron ore. The remainder is greensand, marl, and clayey gravel. The cost of the land, including right of way, was $4,120, and with fencing and preparation, $6,207. Engineering and superintendence cost $1,944. The entire force of labor consists of one man at $300 per annum. The disposal area was seeded with Hungarian grass the first year, which sold standing for $45, but the hay is said to have molded after being housed. An attempt to cure this grass the second year is also said to have failed. Probably if Hungarian grass is the proper crop to raise on this area, ensilage would be the best way to preserve it.[2]

Lawrenceville School.—A system of subsurface irrigation was carried out at this school in 1883. Additions thereto, either by intermittent filtration or by broad irrigation, were proposed in 1893. As to whether the improvements have all been carried out there is no information at hand.[3]

Long Branch.—The winter population of this place does not exceed 7,000 or 8,000, although the population in summer is estimated at from 80,000 to 100,000. Like Atlantic City, this place has granted a franchise for sewerage works to a private company. Chemical-precipitation and mechanical-separation sewage-disposal works were constructed in 1886. Definite information as to results is lacking.[4]

Morris Plains Asylum.—Works for sewage disposal by sedimenta-

[1] Refer to Eng. and Bldg. Record, Vol. XXXVI, July 3, 1897.

[2] Refer to Seventeenth Ann. Rept. New Jersey St. Bd. of Health, 1893.

[3] *Refer to Trans.* Am. Soc. Civ. Eng., Vol. XVI, 1887; Eng. and Bldg. Record, Vol. XVI, 1886 and *Sewage Disposal* in the United States.

[4] *Refer to* Eleventh Ann. Rept. New Jersey St. Bd. of Health, 1887.

A.

B.

SEWAGE DISPOSAL AT PLAINFIELD, NEW JERSEY

A. Main carrier as seen from screen tank (before erection of building); *B.* Distributing channel in bed No. 1.

tion and subsurface irrigation were constructed at this State institution in 1887. The sewage is also dosed with a small quantity of sulphate of alumina. No information is at hand as to whether sewage irrigation could be profitably used.[1]

Plainfield.—The present population of Plainfield is about 13,000. It is chiefly a residence town, a very large portion of the residents being New York City business men. The construction of a water supply a few years ago has had the usual result of emphasizing the need of sewers, but no practical move was made in this direction until 1892. It was then resolved to construct a sewerage system with purification by intermittent filtration and broad irrigation. The construction of the works was begun in 1894 and completed in 1895. The land selected for sewage disposal lies at an elevation of from 50 to 70 feet above mean sea level and can be mostly reached with the sewage of the town without pumping. For one district, however, it is arranged in the future to drain to a pump well, from which the sewage will be raised by automatic electric pumps into a neighboring main, whence it can flow by gravity to the disposal works. The works at Plainfield are of further interest because of a trial there of cultivation of the filtration area. The result of such experiment with indian corn is well shown on Pl. I of Water-Supply and Irrigation Paper No. 3, and on Pl. V of this paper. The flow of sewage is stated at present to amount to from 11,000 to 13,000 gallons per hour at 11 a. m., gradually diminishing from that time to 5,000 gallons at about 4 p. m., and again increasing from 7 p. m. to 11,000 gallons per hour at 11 a. m., as stated. The total flow in twenty-four hours is stated as about 230,000 gallons. The works were constructed under the direction of A. J. Gavett.

Princeton College.—Sewage disposal by broad irrigation was carried out at Princeton several years ago. An average daily flow of sewage of from 7,000 to 10,000 gallons is purified on a sloping field of about 4 acres which has 6 inches of black soil resting upon a homogeneous clay subsoil. The entire area is drained to a depth of 4.5 feet with 2-inch tiles laid 30 feet apart. Sewage is applied from a line of troughs at the upper margin of the field, which are changed from place to place as necessary.[2]

Summit.—Population of the township in 1890, 3,502. Sewage purification by intermittent filtration was first operated in 1892. The disposal area consists of 26 acres of land, of which 10 acres have been laid out. The following bacteriological examination, made by Dr. Richard N. Connolly, bacteriologist to the Passaic River Commission, indicates defects of some sort in these works.

Sample of raw sewage taken December 31, 1896, showed bacilli coli communis very numerous, with a total number of bacteria per cubic centimeter of 1,112,250.

---

[1] Refer to Eleventh Ann. Rept. New Jersey St. Bd. of Health, 1887.

[2] *Refer to Twentieth* Ann. Rept. New Jersey St. Bd. Health, 1896.

A sample of the filtrate from the westerly outlet taken at the same time also showed bacilli coli communis very numerous, with a total number of bacteria per cubic centimeter of 1,098,900.

A sample of the filtrate from the northerly outlet, also taken at the same time, showed bacilli coli communis numerous, with a total number of bacteria per cubic centimeter of 645,440.[1]

Westfield.—This town had a population in 1897 of about 3,000. Sewerage and sewage-disposal works were carried out in 1895 and 1896. The method of disposal is by means of intermittent filtration and broad irrigation. For the former four level beds of about three-quarters of an acre each have been prepared on the highest part of the disposal field, which lies at an elevation of about 80 feet above tide water. The soil ranges from a fine sand to a coarse gravel. For the present 12 acres have been purchased, the portion not included in the filter beds to be used for broad irrigation. Thus far only one man is found necessary to take care of the sewage. The cost of the plant, including land, was $8,435.[2] F. A. Dunham was designing and constructing engineer.

PENNSYLVANIA.

Altoona.—Population in 1890, 30,337; in 1895 the population was estimated at 35,500. The town is located at the base of the Allegheny Mountains, between the elevations, 1,110 and 1,350 feet above tide, and near the head waters of the Juniata River. A sewage-disposal plant, constructed in 1896 and 1897, is designed to purify the sewage of a little less than one-half of the city, which originally drained southward toward Hollidaysburg, the county seat of Blair County, which is situated on the stream originally receiving the sewage a few miles down.

The pollution of the streams receiving the Altoona sewage has been a source of controversy for many years, and in 1882 suits were brought against the city for polluting springs on property about 11,400 feet southward from the city limit. This case was finally decided against the city, and claims of damages, interest, and costs to the amount of $6,555.36 were compromised in 1895 for $2,812.45.

Early in 1895 surveys for a sewage-disposal plant were made by Mr. Harvey Linton, city engineer. Messrs. Rudolph Hering and Allen Hazen were employed as consulting engineers. The details of the Altoona sewage-disposal operations are exceedingly interesting, but too extensive to be gone into here, and we may simply state that the filter beds have a capacity of about 750,000 gallons a day. The land purchased for sewage disposal amounts to nearly 96 acres and cost $12,000. Of this nearly 70 acres are available for sewage purification. The new works were first operated in June, 1897. It is intended to raise corn upon the filter areas.

The specifications for this work, as prepared mostly by Mr. Hazen,

[1] *Refer to Eng. News*, Vol. XXVIII, December 8, 1892, and Sewage Disposal in the United States.
[2] *Refer to Twentieth Ann. Rept.* New Jersey St. Bd. Health, 1896.

present some interesting features. The following is the clause covering grading and embankments:

The general object of this work is to divide up the land into a series of level plats surrounded by embankments, so that each can be separately flooded without disturbing the others, and to prevent the overflow of the land by the stream in time of flood.

. . . . . .

The land should be kept in as loose and porous condition as possible, and to that end all that part of the area to be filled shall be plowed 1 foot deep and left in loose condition before commencing to fill, and the elevations of the filling shall be carried to such additional elevation as may be directed by the engineer, to allow for subsequent settlement to bring all parts of the bed to the same ultimate elevation.

The dividing embankments shall be solidly constructed and rolled if necessary. The stone removed from the land may be largely placed in them if not otherwise required. The outside embankment as low as ordinary high water shall be paved with stones removed from the field in such exposed places as may be directed by the engineer, and the remainder of the slope and top carefully sodded with firm, healthy sods.

Land stones over 4 inches long are to be removed from the surface material as fast as the same is removed and are to be placed in the embankments or along the river to prevent washing, as will best serve the interests of the work, and as may be directed by the engineer.

After removing the stones the surface of the beds shall be carefully graded and smoothed and left in porous condition, free from compacting from wheels or hoofs, and ready for receiving sewage.[1]

Norristown, State Hospital for the Insane.—The total population of this institution is about 1,500, and the total daily consumption of water about 325,000 gallons, or 150 gallons per capita. A sewage-disposal plant, with purification by filtration and broad irrigation, was constructed in 1885. These works are said to have been specially successful.[2]

Reading.—Population in 1890, 58,661. The proprietary process of the Pennsylvania Sanitary Sewage Company was carried out in 1895. It is claimed that the works have sufficient capacity to purify 15,000,000 gallons of sewage a day. The following statement embodies the main points of these works:

The sewage falls by gravity into reservoirs 20 feet in diameter, with coke filters 12 inches deep, through which the sewage first passes in order to screen out the coarser suspended matter. The reservoirs are ventilated by special passages which surround the smoke flue and are expected to discharge foul odors at a height of about 100 feet.

The sewage is drawn from the reservoir into a suction chamber, from which it is pumped to filter beds on the river banks, about a mile from the pumping station. These beds are two-story structures of brick, stone, and iron, 250 by 50 feet, each story being subdivided into ten compartments, making a total of twenty compartments 25 by 50 feet. The filter beds are divided into upper and lower sections, in order to combine aeration with filtration. The filtering material of the upper section consists of fine sand 2 feet thick. After percolating through

[1] *Proc. Eng. Club of Philadelphia*, Vol. XIV, July–September, 1897.
[2] *Refer to Colonel Waring's* Sewerage and Land Drainage, 1889.

this the sewage falls 8 feet to a lower filter, which is formed of various grades and sizes of filtering material. Inlets are provided in which air is to be circulated in the coarser material of the lower section. The effluent water is discharged directly into the river.

The author can not but think that the proposition to purify 15,000,000 gallons of sewage a day by this plant must be intended as a joke.[1]

Wayne.—This place is a suburban village with a population in 1890 of 997. It has grown rapidly, and at present the population is much greater. There are no manufactories. Sewage disposal by surface irrigation was carried out in 1891. The disposal area includes 11 acres of sidehill land, with creek flowing through the middle. A portion of the land is timbered, as shown by fig. 2 in Water-Supply and Irrigation Paper No. 3 and by Pl. VII in this paper.

The sewage flows by gravity to the pumping station, whence it is forced to distribution wells on adjacent highlands, from which it flows down the sidehill through heaps of broken stone. At various places below cinder banks are formed. A very efficient filtration is said to be obtained by this process in course of a few hundred feet below. In 1892 the average daily quantity of sewage was established at about 200,000 gallons. During that year five crops of grass were raised on the irrigated land.[2] The effluent water is discharged into the river.

WEST VIRGINIA.

Weston, State Insane Hospital.—Plans for sewage disposal by chemical precipitation, supplemented by broad irrigation, on about 27 acres of land on which it was intended to grow crops were prepared in 1891. In view of the sewage farming offering an opportunity for employing a considerable number of harmless insane in light outdoor labor, it was believed that this project was an especially feasible one. Thus far, however, the West Virginia legislature has not appropriated the $40,000 necessary in order to carry out the project in full detail.[3]

TEXAS

Paris.—Population in 1890, 8,254. This town is finely located in northwestern Texas, about 14 miles from Red River, at an elevation of 588 feet above tide water.

Sewers were constructed in 1894, discharging into a small stream known as Bankers Creek. Soon after constructing a main sewer, farmers on this creek below the outfall brought suits for damages aggregating $15,000. Trial of one of them resulted in a verdict against the city of $900.

The suits had the effect of forcing attention to some other means of sewage disposal, and in the early part of 1897 a system of disposal

[1] *Eng.* and Bldg. Record, Vol. XXXI, May 4, 1895.

[2] *Refer to the* Am. Arch., July 2, 1892; Eng. News, Vol. XXVIII, November 3, 1892; and Sewage *Disposal in the United States.*

[3] *Refer to Jour. of* W. Va. House of Delegates, February 18, 1891.

by intermittent filtration was begun. For this purpose the city purchased 10 acres of land for $750, of which 5½ acres have been laid out. The cost of constructing 5½ acres of filters is given at $3,730. The present daily flow of sewage is from 35,000 to 40,000 gallons. The sewage-disposal areas are cared for by one man, who is paid at the rate of $420 a year. It is proposed to crop with alfalfa in 1898.[1]

San Antonio.—Population in 1890, 36,673. The city purchased 530 acres of land in 1895, with the expectation of establishing a sewage farm. Information is not at hand as to the present state of the enterprise.

Temple.—Population in 1890, 4,047. It is understood that this place has recently granted a franchise to a private company to construct and operate sewerage and sewage-disposal works.

OHIO.

Alliance.—Population in 1890, 6,707. A chemical-purification plant estimated to cost $18,400 has either been recently constructed or is about to be constructed at this place. As to just the present state of the project, no information is at hand.

Canton.—Population in 1890, 26,189; present population estimated at about 35,000. The outfall sewer originally discharged raw sewage into the main branch of the Nimishillen Creek. Complaints were made several years ago by the owners of land abutting on the creek below the outlet of the sewer, thus bringing the question of sewage purification before the city authorities, who acquired 28 acres of land at and about the sewer outfall for the purpose of providing some method of purification. Chemical treatment was finally decided upon as adequate to the requirements, although the relative cost of chemical purification versus land treatment does not appear to have been gone into. The works were constructed under the supervision of L. E. Chapin, city engineer.

The plant was first started in May, 1893, since which time all the sewage of the city has been treated. The following analyses, made by Dr. Curtis C. Howard, in which the results are stated in parts per 100,000, indicate the degree of purification effected. No. 1 is the average of 24 samples of the raw sewage, and No. 2 the average of 24 samples of effluent taken at the same time as No. 1.

Analyses of raw sewage and of effluent, at Canton, Ohio.

[Parts per 100,000.]

	No. 1.	No. 2.
Free ammonia	0.894	0.676
Albuminoid ammonia	.326	.112
Chlorine	2.14	2.05
Total solids	49.3	41

[1] *Refer to Eng. News*, Vol. XXXVIII, August 12, 1897.

The cost of the sewage-disposal plant at Canton to March 15, 1897, is as follows:

Land	$4,100.00
Construction	27,215.22
Material	14.70
Total	31,329.92

The cost of maintaining the works in 1896 was as follows:

Labor	$1,630.00
Coal	356.76
Lime	739.63
Alum	502.75
Oil and waste	33.01
Filter sacks	46.68
General repairs	61.91
Miscellaneous	44.50
New smokestack	137.00
Total for year	[1] 3,552.23

Fostoria.—The population in 1890 was 7,070. Plans for sewage disposal by land purification were prepared in 1895. As to whether they have yet been carried out no information is at hand. The conditions are such as to require pumping to the disposal field. Cost of disposal works is estimated as follows:

24½ acres of land, at $150	$3,675.00
Preparation for beds, including underdrainage, grading, force main, main carrier, and effluent drain	6,425.00
Reservoir, sludge and pump well, necessary buildings, boiler, and pumps	14,920.00
Total	25,020.00

Running expenses with coal as fuel are estimated at about $1,843 per year, without taking into account the profit to be realized from the sale of produce raised on the filter areas.[2]

Oberlin.—This place, which is the site of Oberlin College, had a population in 1890 of 4,376. A sewerage system, with disposal by broad irrigation, was constructed in 1893. The irrigation field includes 15 acres, purchased at a cost of $1,500. In 1894 the daily flow of sewage varied from 35,000 to 40,000 gallons. In addition to broad irrigation, 9 filter beds have also been prepared, of which 3 are devoted to raising woodland rye grass, 3 to alfalfa, and 3 to corn and garden vegetables.

The soil, which consists of yellow clay underlain at a few feet by stiff blue clay, is not specially favorable for sewage disposal. As to the best methods of treating such a soil, reference may be made to a description of the sewage farm at Wimbledon, England, in Water-Supply and Irrigation Paper No. 3. The Oberlin sewage farm is also especially interesting because of the small amount spent in preparation.

[1] Refer to Canton Municipal Reports and Tenth Ann. Rept. Ohio St. Bd. Health, 1895.
[2] Refer to Tenth Ann. Rept. Ohio St. Bd. Health, 1895.

The following are the figures:

6,000 feet of agricultural tile	$256
Tool house	55
Grading and incidentals	166
Small pump and tank	42
Land	1,500
Total	2,019

Some reports as to a nuisance at the Oberlin sewage farm having gained circulation, a committee of the Ohio State board of health inspected it in July, 1895. In their report the committee say that no just cause exists for complaint in respect to the Oberlin sewage farm.[1]

Sewage disposal plants are also reported as projected at Bowling Green, Cambridge, Chillicothe, Columbus, Glenville, Greenville, and Mansfield, Ohio.

MICHIGAN.

Wayne County Poor Farm.—A chemical precipitation plant, costing about $10,000, was first operated at this institution in 1896. Cost of operation, including chemicals and attendance, is estimated at about 80 cents per capita per annum for a population of 800.[2]

St. Johns.—Population in 1890, 3,127. A land-purification project has been proposed.

ILLINOIS.

Pullman.[3]—This farm was laid out in 1880 and 1881 by the Pullman Palace Car Company as a means of purifying the sewage of the model town of Pullman, which is about 14 miles south of Chicago and at present is included within the limits of that city. The population in 1893 was 11,000. The only body of water in the vicinity which could possibly receive crude or raw sewage is Lake Calumet, a lake about 3 miles long, 1½ miles wide, and from 1 to 8 feet deep. The extreme shallowness of this lake, as well as the absence of current, renders it very undesirable as a place for disposing of crude sewage, and the country about Pullman is so flat that it was necessary to pump the sewage to the area selected for farming purposes. The area originally selected comprises about 1,500 acres, though thus far only a small part of this tract has been used. In 1892, with an average daily flow of sewage of about 1,850,000 gallons in twenty-four hours, the land in actual use did not exceed 140 acres.

This farm is of special interest in the present connection, because its original design recognized the principle of having filter areas on which any surplus sewage not needed for the best results in irrigation could be purified. It should not be overlooked that reports have frequently come from Pullman that the sewage was often allowed to flow into

[1] Refer to Ninth Ann. Rept. Ohio St. Bd. Health, 1894 and 1895.
[2] Refer to Eng. and Bldg. Record, Vol. XXXIV, July 11, 1896.
[3] The use of this farm for sewage disposal has recently been discontinued.

Lake Calumet in nearly its natural condition. On this point it may be remarked that the projectors have probably felt that little harm was done to anybody so long as no nuisance was created in the lake or along its margins. The fact that a portion of the ice supply of Chicago was taken from Lake Calumet during the winter has been considered with indifference. Owing, therefore, to either inefficient or indifferent management, the Pullman farm can not be referred to as a specially successful example of sewage purification, although, from the statements made, the utilization has justified in a commercial way the expectations of its projectors. The farm is stated to have yielded in some years a net profit of from 8 to 10 per cent. The designing engineer, Benezette Williams, of Chicago, is entitled to the credit of having made his design sixteen years ago in line with what is universally considered to be the best practice of the present day.

The soil at Pullman is the ordinary prairie black alluvium, about 1 foot in depth, and mostly underlain by a clay subsoil. It is therefore not the most favorable for utilizing a large volume of sewage on a small area. From what has been learned in regard to this farm, it is the author's opinion that a considerably larger area should be used. With this improvement, as well as some additions to the filter area in order to afford sufficient relief in wet weather, it is believed that sewage farming can be made very successful at Pullman, from the points of view of both purification and utilization.

The underdrains are of common agricultural tile, laid about 40 feet apart to an average depth of about 4½ feet. The most satisfactory crops are cabbage, cauliflower, celery, and other vegetables common to market gardening. For some reason the raising of potatoes has not been successful on this farm. Italian rye grass is also reported as unsuited to the conditions, but whether the difficulty is due to climate or soil, or both, is unknown. Very little live stock is kept, because market gardening for the Chicago market is more profitable. During the growing season about forty laborers are employed at the current rates of common labor in the vicinity of Chicago.

The distribution of sewage at Pullman is effected by the pipe-and-hydrant system. For this purpose vitrified tile pipes, tested by water pressure before laying, have been successfully used.

The foregoing short account of the Pullman sewage farm is given here, not because it is an example of the best work in sewage utilization, but because it was the first large sewage farm in the United States, and because, further, it was designed in accordance with the now universally recognized principle of intermittent filtration relief areas. In view of the large experience gained elsewhere, failure to purify the sewage efficiently at all times must, as has already been pointed out, be attributed to the management rather than to the method. Under this head it may be remarked that thus far the American farms, generally speaking, have not realized their full agricultural capacity

by reason of defects in management. We need to develop here, as there have been developed abroad, a class of sewage-farm managers who, having made a specialty of this method of farming, shall be not only informed as to details but qualified to meet emergencies.

As regards sewage-purification works owned and operated by towns, it may be said that the entire separation of the management of the works from local politics will be the first necessary step toward increased efficiency.

World's Fair.—The sewage of the World's Columbian Exposition, in 1893, was purified by mechanical precipitation before discharged into Lake Michigan. The plant assigned for this purpose embodied the method of vertical tanks, an illustration of which may be found at page 556 of Sewage Disposal in the United States.[1]

This plant was placed in charge of Mr. Allen Hazen, who made extensive studies as to its efficiency. The daily quantity of sewage treated varied from about 6 million to 9 million gallons. The total cost of treatment per million gallons was $38.09. The works cost about $55,000, of which $20,000 was for the building.

The cost of the chief precipitant used varied from $20.40 to $47 per ton. Mr. Hazen's results are given in considerable detail, but can not be further gone into here. On the whole his report is adverse to the efficiency of the process.[2]

WISCONSIN.

Menominee Valley.—The sewage-disposal plants in Menominee Valley have been described on page 24, in connection with the discussion of purification of manufacturing wastes. They include Schmidt's Curled Hair Works, McBride's Sanitarium, the National Soldiers' Home, the public institutions of Milwaukee County, and the Miller Brewing Company.[3]

MINNESOTA.

Rochester Hospital for the Insane.—Sewage disposal by chemical precipitation supplemented by intermittent filtration was first operated at this State institution in 1890. The population of the hospital is stated at about 1,050 persons, with a total sewage flow of 60,000 gallons a day. The sewage first flows to tanks of sufficient capacity to handle 75,000 gallons a day, from which, after treatment with chemicals, the effluent is pumped to a filtration area with about 16 inches of surface loam, below which is sand and gravel. This area is underdrained with 3-inch tile laid in lines about 25 feet apart. In spite of the low winter temperature of Minnesota, it is stated that no trouble has been experienced in securing sufficient winter purification. The sewage probably

[1] Additional illustrations of this plant are given in Eng. News, Vol. XX, August 3, 1893.

[2] The full results of Mr. Hazen's studies of the World's Columbian Exposition sewage-disposal plant may be found in the Twenty-fifth Ann. Rept. Mass St. Bd. Health, 1893.

[3] Refer to Eng. News, Vol. XXXIII, June 13, 1895, and Sixteenth Rept. Wis. St. Bd. Health, 1895-96.

reaches the precipitation tank in cold weather with a temperature of about 45° F.[1]

NEBRASKA.

Hastings.—Population in 1890, 13,584. Stated present population, 15,000 to 18,000. A sewage farm of about 70 acres was carried out in 1891 or 1892. The soil of this farm is open and porous, and, so far as known, has not been underdrained. No information is at hand as to the success experienced in cropping.[2]

COLORADO.

Colorado Springs.—Population in 1890, 11,140. A sewerage system was carried out in 1888 and a sewage farm in 1889. Soon after opening the sewage works a ranchman living 2 miles below the outfall, which is into a small stream with a normal flow of about 50 cubic feet per second and at times with no flow at all, instituted injunction proceedings, claiming that his well was so polluted as to render the water unfit for drinking, and that the water in his irrigating ditch, of which the head gate is about three-quarters of a mile below the sewer outfall, was so foul that stock would not drink it. The matter was arbitrated and the suit withdrawn, the city paying all costs of proceedings and agreeing to divert the sewage and utilize it for irrigation.

A contract was then made between the city and the owner of a tract of about 35 acres of land upon which the sewage could be conveniently diverted by gravity, the city agreeing to pay $300 annually for five years, and the owner on his part agreeing to use the sewage for irrigation purposes in such manner as he deemed best, provided, however, that the sewage be prevented from flowing directly into the creek, and provided further, that if the method of irrigation was not satisfactory to parties on the stream below, the city should take possession of the land and use such methods as it thought best. At the expiration of the contract the city was to have the option of buying the land at a stipulated price. Whether or not this has been done is not known. The irrigation, however, is said to have been very successful. In 1892 sewage was used on 25 acres—15 acres in meadow and alfalfa and 10 acres in vegetables. The crops produced have been very large, and owing to close proximity to market the farm is said to be a paying investment.

As illustrating the demand for water in this section, it is stated that farmers several miles below the sewer outfall, seeing the sewage farm well supplied with water at times when their farms were suffering from shortage, have threatened to enjoin the city from using the sewage for this purpose, and thus compelling it to turn the sewage into the stream for their benefit.[3]

Leadville.—Population in 1890, 10,634. The sewerage system of this

[1] Refer to Sixth Biennial Rept. of Trustees, 1890, Eng. and Bldg. Rec'd., Vol. XXIII, and Sewage Disposal in the United States.

[2] *Refer to Eng. News*, Vol. XXIX, March 9, 1893, and Sewage Disposal in the United States.

[3] *Refer to Eng. News*, Vol. XXIX, February 23, 1893, and Sewage Disposal in the United States.

A.

B.

SEWAGE IRRIGATION AT SALT LAKE CITY UTAH

A, Main ditch, outlet gate, and field of corn; *B*, Lateral ditch and method of irrigating corn.

place was built in 1886 by the Leadville Sewerage Company. The original sewage-disposal works as constructed by this company included a sand and gravel filter about 6 or 7 feet deep, with a surface area of 24 square feet. Whether this extensive sewage-disposal plant is still in use is not known.

Trinidad.—Population in 1890, 5,523. A sewerage system was first operated in 1892. Sewage is first received in a settling tank 50 feet long, 5 feet wide, and 4 feet deep, from which it flows upon land. To the end of 1892 about $1,200 had been expended in preparing the irrigation area to receive sewage. Blue grass was originally proposed as a crop on the farm, because it was supposed that this grass would stand more frequent irrigation than any other crop. A private party owns the irrigation area, on which the city delivers the sewage and pays $500 a year to the owner.

MONTANA.

Helena.—The population in 1890 was 13,134. A sewerage system was first operated in 1889. At this place the sewage is utilized only in the growing season. During times when the ground is so frozen that there is no absorption, sewage flows in natural channels upon the surface. The lower end of the outfall sewer connects directly with the distributing ditches of the farm, which includes 40 acres of land, purchased for $6,100. It is leased at an annual rental of $200. Crops raised are vegetables and nursery stock.

UTAH.

Salt Lake City.—Population in 1890, 44,843. A sewage farm was carried out at this place under the direction of F. C. Kelsey, city engineer, in 1895 and 1896. The daily flow of sewage is stated at 4,000,000 gallons. In addition storm water to the amount of 15,000,000 gallons per day may be expected at different times. The land acquired by the city amounts to 183 acres, of which 80 acres have been plowed and are under cultivation in 1897. It fronts on the Jordan River and the main ditch is extended to the stream, so that when the outfall sewer is running full the flow can be diverted directly to the river without passing over the land. Settling basins to the extent of 30 acres have been prepared.

Sewage was first delivered on the farm on July 11, 1896. No crops were planted that year except 5 acres of corn for fodder. It is intended to improve the farm by erecting buildings, planting shade trees along the highways, and an orchard of 6 acres, which is too high for irrigation, but thus far the money has not been furnished for these improvements.

The results of this extensive sewage irrigation at Salt Lake City will be looked for with interest, the more especially as, when the sewage farm was first proposed, it was thought that neighboring farmers would be willing to take the sewage for irrigation, because all land at Salt Lake City necessarily requires irrigation. The farmers, however, refused to take it, *mainly, it is said,* from ignorance, thinking it would damage the

crops, clog the ground, and be otherwise injurious. It was for this reason that the city embarked in sewage farming.[1]

The total cost of the sewage farm to January 1, 1897, was: Land, $14,763.45; plowing, $273.42; preparation of surface, etc., $3,452.64; total, $18,489.51.

The total cost of handling the sewage for the last six months of 1896 was $683.36. Assuming an average of 4,000,000 gallons a day, this would be at the rate of about 93 cents per million gallons handled for the six-months period. Probably when the sewage farm is in full operation the cost will be greater, but it will be met by sale of crops.

ARIZONA.

Phœnix.—Population in 1890, 3,152. The waterworks, sewerage, and sewage disposal at this place are all owned by the Arizona Water Works and Sewage Company. According to the information at hand, 160 acres of land have been provided for a sewage farm, on which sewage is largely used for irrigating garden truck by Chinese gardeners. The results are understood to be satisfactory. A view of an irrigated field will be found in Pl. VII.

CALIFORNIA.

Fresno.—Population in 1890, 10,818. Sewers were constructed in 1889. In 1890 a contract was made with Alexander McBean, of Oakland, to take the sewage and purify it by irrigation, the city paying therefor $5,000 per year. Mr. McBean purchased 80 acres of land at the outfall, and during the first year the sewage ran upon it without special attention or care, except when some neighbor saw fit to take it for irrigation. The second year distribution ditches were constructed and the land leased to Chinese gardeners for vegetable gardens and vineyards. The land is now all under cultivation with the various kinds of vegetables commonly in the market, such as potatoes, yams, parsnips, lettuce, celery, beans, peas, and corn. Statements as to the amount of sewage are lacking.[2]

Los Angeles.—Sewage irrigation at Los Angeles is of considerable interest, because the most extensive sewage farming by private owners in the United States is practiced there. The population, according to the census of 1890, was 50,395. In 1894 a conservative estimate placed it at 70,000.

Los Angeles is built upon both sides of a torrential stream, Los Angeles River, at a point about 20 miles from its mouth, and at a distance of about 12 miles from the Pacific Ocean. Sewage irrigation here was originally carried on by a private company, which took the sewage as delivered from the main outfall sewer and conveyed it by ditches to an area on which it was utilized in agriculture as required on various small tracts, the surplus being allowed to flow away into

[1] Refer to the Salt Lake City Municipal Rept. for 1896, and Eng. News, Vol. XXXVII, March 13, 1897.
[2] *Refer to Sewage Disposal in the United States.*

A. SEWAGE IRRIGATION AT PHŒNIX, ARIZONA.

B. SIDEHILL IRRIGATION AT WAYNE, PENNSYLVANIA

the stream unpurified. In the course of time the area originally irrigated became city property, and a large portion of it having been sold for residence sites, many of the landowners objected to sewage irrigation and obtained injunctions restraining its use in this way. According to the statements made there was at one time considerable nuisance from the sewage irrigation. The ditches were not kept clean, and under the hot sun of southern California they gave forth a strong odor. The objection was also made that sewage irrigation was detrimental to the health of the community, but so far as can be learned no facts have ever been produced to support this objection.

The sewage has been mostly used in growing vegetables for the Los Angeles market, and, according to the statements made, its commercial value for such purpose has been considerable.

As the result of several years' discussion the city of Los Angeles has recently constructed a main outfall sewer leading to the Pacific Ocean, along the line of which numerous valves have been placed, as is done at other places where successful sewage irrigation has been practiced, not only for drawing off the sewage for use in agriculture, but for restricting its use to such times as it may be needed.

The soil for a considerable portion of the distance along the main sewer is light and sandy and especially adapted to successful sewage purification, and it is believed that a considerable use of sewage for this purpose will be developed soon.

In 1895 and 1896 a total area of about 1,550 acres was irrigated along the line of the new outfall sewer, the amount of sewage used upon this acreage varying from 8 to 10 cubic feet per second. An area of 800 acres near the city is mainly devoted to raising vegetables. The balance was planted in corn and other fall crops, among which were about 100 acres in potatoes and cabbage. According to the reports of the city engineer for the years 1895 and 1896, the following revenue was realized from the sale of sewage:

Revenue realized from the sale of sewage at Los Angeles, California, in 1895 and 1896.

1895.		
Receipts from the sale of sewage		$3,278.00
Maintenance:		
Salary of superintendent	$1,000.00	
Assistance and labor	625.47	
Repairs, etc	512.50	
		2,137.97
Net revenue		1,140.03
1896.		
Receipts from sale of sewage		$4,009.50
Maintenance:		
Salary of superintendent	$1,200.00	
Assistance and labor	1,675.40	
Repairs, etc	184.80	
		3,060.20
Net revenue		949.30

In 1883 the city of Los Angeles entered into contract with the South Side Irrigation Company, obligating itself to give to said company all the sewage flowing through the San Pedro street sewer, amounting to about 3.58 cubic feet per second. In October, 1895, a second contract was made with the same company, whereby the city agreed to deliver until the expiration of the former contract an additional quantity of 2.39 cubic feet per second, or a total until 1903 of a little less than 6 cubic feet per second. In consideration of the last amount the company has laid a 24-inch cement pipe, 6,000 feet of which lies within the city limits and which will become the property of the city in 1903. The company also has the right to use an additional quantity of sewage by paying the prevailing price for it, which during the height of the irrigation season, in April and May, has been placed at $8 per twenty-four hours for 2 cubic feet per second. The rate is also made $5 for a day run of twelve hours and $3 for a night run, to suit the convenience of the irrigators.

The sewage now controlled by the South Side Irrigation Company is mainly used for irrigating about 2,200 acres of land devoted to vegetable gardening. The sewage is distributed throughout the land irrigated by ordinary open-earth ditches. This land now pays an annual rental of $12 per acre for river-water irrigation and $18 per acre for sewage irrigation, or $6 per acre in favor of the sewage.

On sandy soil with good subdrainage the South-Side Irrigation Company now irrigate successfully during the whole year. Experience has shown that when not actually needing the sewage for irrigation it is better to flood the land than to permit the sewage to go to waste.[1]

Pasadena.—Population in 1890, 4,882. Sewers were constructed from 1887 to 1891, and a city sewage farm was brought into operation in 1893. Its soil is a light sandy loam, with good natural drainage, and includes at the present time an area of 800 acres. The flow of sewage is estimated at about 3 cubic feet per second. According to statements of Mr. Burr Bassell, of the 300 acres actually owned by the city, 140 are used for raising grain and have never been irrigated. Sewage has been used upon the remaining cultivated area as follows: Walnut orchard (4 years old), 60 acres; alfalfa, 25 acres; corn, 25 acres; vege-

[1] In a paper by Mr. Burr Bassell, deputy city engineer of Los Angeles, on the Operation of the Los Angeles outfall sewer and sewage irrigation, which appeared in the Journal of the Association of Engineering Societies for August, 1897, many additional interesting details may be found. In the discussion of Mr. Burr Bassell's paper, James D. Schuyler, a leading California irrigation engineer, pointed out that, in view of the conspicuous success achieved in sewage farming abroad, the policy of constructing an outfall sewer to the ocean must be regarded as questionable, particularly in southern California, where the soil and climatic conditions are all extremely favorable for successful sewage farming. Mr. Schuyler expressed the hope that in the future growth of Los Angeles, when the capacity of the present outfall has been reached, extension of the system would be made in the line of utilization by irrigation and the establishment of a city sewage farm rather than in the duplication of the present sewer to the ocean. Mr. Schuyler gave a large amount of valuable information as to the comparative cost of river water and sewage irrigation in the vicinity of Los Angeles.

In addition to Mr. Bassell's paper, refer to Report of a board of engineers upon the disposal of the *sewage of Los Angeles*, etc., 1889, Eng. News, Vol. XXIX, February 23, 1893, and Sewage Disposal in *the United States*.

tables, potatoes, etc., 50 acres. The farm is self-sustaining, cost for maintenance and revenue being each about $2,500 per annum.

The experience at Los Angeles and Pasadena seems to be conclusive that there is no valid objection to the use of sewage for the irrigation and growth of all kinds of agricultural products. It is apparently a question merely of proper application. As showing the immense advantage of sewage disposal in the vicinity of Pasadena, it may be mentioned that bare land is worth at least $100 an acre without water, while watered land is held at about $600 an acre. The Pasadena sewage farm originally cost $125 an acre, or a total, including some extra expenses, of about $40,000.[1]

Redding.—Population in 1890, 1,821. A separate sewerage system was built by the town in 1889. Bassett & Rouhey, of Sacramento, the contractors for the sewage work, entered into an agreement to dispose of the sewage for forty years. For this purpose they purchased a tract of about 100 acres of land and prepared a portion of it for the utilization of sewage by irrigation. The area selected is comparatively level, with the soil a sandy and gravelly loam 4 to 6 feet deep, underlain by gravel. The sewage is applied directly to the land, either by running in furrows between rows or by spreading over the surface, according to the special requirements of each crop. The chief crops raised are corn, asparagus, potatoes, turnips, beets, and other garden truck. It has also been used for orchards and nursery stock. When used in this manner the trees were irrigated between the rows.[2]

San Luis Obispo.—Population in 1890, 2,995. Plans for sewage disposal by broad irrigation and intermittent filtration were prepared about 1888. No information is at hand as to whether or not they have been carried out.[3]

Santa Rosa.—Population in 1890, 5,220. Complaints of the pollution of the creek into which the sewage of the town was originally discharged, followed by a lawsuit, led to the purchase by the city of 18 or 19 acres of land on which the sewage has been purified since about 1889. The farm is leased to tenants, who take care of the sewage as rental, using it for gardening purposes.

Stockton.—Population in 1890, 14,224. The sewage of this place is stated to be used for irrigating purposes, but to what extent is not known.

DOMINION OF CANADA.

Berlin.—Berlin is the county seat of Waterloo County, and according to the census of 1891 had a population of 7,425. The chief industries of the town are manufacturing establishments, among which are included a large tannery and a glue factory. A public water supply

---

[1] Refer to Burr Bassell's paper in Jour. of Assoc. of Eng. Socs., Vol. XIX, August, 1897; and to Sewage Disposal in the United States.

[2] Refer to Sewage Disposal in the United States.

[3] Refer to Colonel Waring's Sewerage and Land Drainage, 1889.

was built in 1888. Construction of sewers began in 1892, and in March, 1893, the actual quantity of sewage was about 50,000 gallons per day, diluted by some 75,000 gallons of subsoil water, making a total of 125,000 gallons in all. A sewage farm was completed in the fall of 1892 and first brought into use in 1893. In January, 1893, the temperature of the sewage as it arrived at the farm was found to be 48° F. The farm is located a mile south of the closely built-up section of the town. A stream, with a summer flow of about 200 cubic feet per minute, passes through the farm and receives the effluent.

The field purchased by the town for sewage-disposal purposes has an area of 20 acres, about an acre of which is included in the stream. The total area aside from the stream can easily be used for sewage farming by slight grading, as the greatest elevation is only 3 feet above the end of the outfall sewer. The surface soil is loam or sand, and the subsoil is of gravelly clay.

The designing engineer recommended that Italian rye grass be raised on the two north divisions of the farm, and on the remainder mangolds and other root crops, onions, oats, and clover. On the lowlands bordering the stream it is proposed to raise osiers, they having been raised with profit in Brantford, where dependence is had upon the freshets of the Grand River for irrigation. It is stated that the farm is suitably arranged for every known system of irrigation and every variety of crops. This farm is interesting from the fact that, as at the Croydon-Beddington farm, no settling tank has been provided, preference being given to the application of the solid matter directly to the land, where it can be dug in or raked off, as may be found most desirable. When in the course of time the amount of sewage has increased so that it exceeds the capacity of the farm for disposal by broad irrigation, it is proposed to construct filter areas at the south end, where gravel may be obtained within reasonable distance. The 20 acres purchased cost $2,000, and the cost of grading and underdraining the 7 acres already prepared was about $3,000, the underdraining and grading having cost at this rate a little over $400 per acre.

The results of cropping at Berlin are stated to have been substantially as follows: In 1894, 2,000 cabbage plants were set on about one-half an acre of land, of which 1,500 grew to marketable heads and were sold at the usual market price of 3 cents each. The cabbages were irrigated regularly and grew well. No attempt was made to hoe around them or hill them up, the surface of the ground being left entirely smooth.

The cultivation of Italian rye grass at Berlin does not seem to have been successful. The rye-grass seed was sown broadcast and soon came up, covering the ground with a mass of roots which are stated to have very much retarded the absorption of sewage. This experiment was made on a filter bed in a manner quite different from the ordinary practice of rye-grass cropping abroad. A chief difficulty of this trial

appears to have been an improper preparation of the land, depressions being left into which the sewage settled and which soon became foul. Abroad, on the contrary, this crop is not raised on beds at all, but on land so prepared as to avoid pondage of the sewage; besides this, the amount of sewage applied is regulated to the necessities of the crop and to the quality of the land, whereas at Berlin, so far as can be learned, the sewage application was made without special reference to the conditions. Under these circumstances the failure at Berlin can apparently have no other significance than to illustrate an erroneous method of procedure.

In 1895 a large area was devoted to cabbage, but by reason of a late frost many of the plants were killed. Later the cabbages were reset and the seeds of onions, carrots, mangolds, corn, and beans were planted. These crops are stated to have all done well except the corn and beans. The following is a detailed statement of the sale of produce from the Berlin farm in 1895, as compiled from the proceedings of the committee of the town council in charge of the farm:

August, 1,217 heads cabbage, at 2½ cents	$30
September, 1,771 heads cabbage, at 2½ cents	44
October, 2,272 heads cabbage, at 2½ cents	57
October, 134 bushels carrots, at 15 cents	20
October, 648 bushels carrots, at 15 cents	97
Corn (failure)	7
Pease (not irrigated to any extent)	15
November, 1,350 heads cabbage, at 2½ cents	34
Total	304

In addition to the foregoing, 150 kegs of sauerkraut, of 50 pounds each, were made from cabbage not sold in the head. These were held at $3 per keg.

The mangolds yielded 648 bushels from one-half an acre, giving at this rate the enormous yield of 1,296 bushels per acre. Those raised were easily sold to farmers for feeding stock. The experience, so far as gained at Berlin, indicates that mangolds will stand very heavy dosing of sewage.[1]

In 1896, complaints having been made by the local board of health of Waterloo Township to the effect that the sewage of Berlin was creating a nuisance in Berlin Creek, the matter was investigated by a committee of the board, which reported in effect that the farm had been managed on more economical lines than was desirable, such misplaced economy being the chief cause of the nuisance complained of. From this point of view the committee of the Provincial board suggested that the town council and local board of health of Berlin be instructed to adopt sufficient and adequate means whereby the whole sewage of the town could be applied on the beds in succession and completely purified before flowing to the creek. After making several

[1] *Refer to Eng. News, Vol. XXIX, April 6, 1893, and Vol. XXXV, June 11, 1896; and to Rept. Prov. Bd. Health, 1896.*

other recommendations the committee concluded its report with the statement that the farm could be made productive without there being any reasonable ground for complaint.

Hamilton.—Purification works by chemical precipitation have been approved for this place by the Provincial Board of Health. In order to insure efficient operation, a committee of the Provincial board which examined the plans in October, 1896, reported that in its opinion the management of the works, when constructed, should be left to the care and supervision of the city engineer, who should also be made responsible for the subordinate engineer in charge. The committee believed that with such management an effluent can be poured into Hamilton Bay of a purity compatible with the best interests of the public health. The works are under construction at the present time.

London.—It is understood that plans for sewage disposal by intermittent filtration are about to be carried out at this place. The land selected for the disposal fields is sandy and stated to be well adapted for the purpose.

London Hospital.—The works at London Hospital were constructed in 1888 and 1889 under the general direction of the architect in chief of the Ontario department of public works.

Previous to the construction of the disposal works the sewage from the hospital debouched into a small stream tributary to the south branch of the Thames River, which flows through the city of London. The drainage area of this stream is so small that it becomes nearly dry during the summer. The population of the hospital is over 1,000, and the daily amount of sewage 60,000 gallons. It became necessary, therefore, that some method of disposal be used other than of allowing the flow of raw sewage into the brook. The plan as carried out involves the flow of sewage into a receiving tank, from which it is pumped, as required, onto the disposal area of 30 acres, situated not far away. The general method is intermittent filtration, supplemented by broad irrigation. For this purpose an area of about 5 acres at the highest portion of the disposal area was leveled and laid out in absorption ditches. The balance of the field has been provided with a main carrier and distribution ditches, for use as an irrigation area whenever the filtration area is overworked or whenever during the growing season the sewage can be profitably utilized thereon for crops.

The filtration area is divided into three sections, onto each of which the sewage is run for one day, thereby giving two days' rest for a section after each application. The details of this work may be found in various publications in regard to sewage disposal.

Ontario Agricultural College.—Sewage purification by treatment with porous carbon supplemented by filtration through land was carried out at this institution in 1889. The daily amount of sewage averages 3,000 gallons. On its way to the settling tank it receives about 12 pounds of ferozone precipitant daily, and is allowed to settle for twenty-four

hours. The effluent is discharged on the filtering ground, which consists of a flat bed of gravel, clay, and loam, 100 by 75 feet in area, divided into two beds, each of which receives on alternate days the sewage for twenty-four hours. These beds are underdrained with tile drains, laid to a depth of from 2½ to 3 feet.

Waterloo.—Waterloo is located 2 miles south of Berlin, the town described previously, with which it is connected by a tramway. The population in 1891 was 2,941. Sewage-disposal works by intermittent filtration were constructed at this place in 1895. For the final works provision has been made for ten filter areas. The soil is sand and gravel. Each bed is 132 by 200 feet, giving an area of 26,400 square feet per bed. This area also includes the embankments, so that the actual purification area is somewhat less. As at Berlin, the sewage will be delivered upon the land without a preliminary screening out of the solid matters, the experience so far gained at the neighboring town of Berlin having indicated that there is little or no necessity for screening. Up to April, 1896, the total expenditure was about $2,000. The cost of the land is stated at about $80 per acre.

Concluding the foregoing account of sewage-disposal plants in the United States and Canada, we wish to state again that in such extreme condensation as is needful in a paper of this kind many important details are necessarily omitted. Complete descriptions of these works would require several hundred pages.

PUBLICATIONS RELATING TO SEWAGE UTILIZATION AND DISPOSAL.

The following books, pamphlets, reports, etc., are in the author's collection and have been consulted frequently in discussing questions of sewage utilization and disposal. There is no pretense to completeness in this list, including, as it does, only such as have been brought together without special effort. The books here included are a fairly good beginning for a library of sewage utilization and disposal and some of the more important cognate questions.

Adams (Julius W.). Sewers and drains in populous districts. 8vo. New York, 1880.

Contains short chapter on sewage disposal.

American Public Health Association, Proceedings of.

American Society of Civil Engineers, Transactions of.

Association of Engineering Societies, Journal of.

Association of Municipal and Sanitary Engineers and Surveyors, Proceedings of. Vols. I–XXI.

Contain a large number of valuable practical papers on sewerage, sewage disposal, and river pollution in England, as prepared by the engineers of the various works. Many of these papers are indispensable to anyone who wishes to study the question from a practical point of view.

Austin (Henry). Report on the means of deodorizing and utilizing the sewage of towns. Paper; 8vo. London, 1857.

Discusses a large number of the more important questions in sewage utilization in such a way as to be of great value to the student of the present day.

Backhouse (Benjamin). An account of Liernur's sewerage system in its present state of development based upon personal inquiry. By the chairman of the city of Sydney Improvement Board. Pamphlet; 8vo. London, 1887.

Baker (Henry B.). Report on best method of disposition of excreta and waste matters, at Harbor Point, Michigan.

Baker (M. N.). Sewage purification in America. Paper; 12mo. New York, 1893.

Baker (M. N.). Sewerage and sewage purification. 16mo. New York, 1896.

Baumeister (R.). The cleaning and sewerage of cities—sewerage, sewage disposal, street cleaning. Translation; 8vo. New York, 1891.

A concise statement of the German views on sewerage, sewage disposal, and street cleaning. Contains illustrations and is especially valuable for the reader who wishes to survey the whole field of the German view without reading a large number of works. As remarked in the introduction, prepared by Rudolph Hering, the American reader should remember that this work was prepared primarily for German engineers and for the conditions prevalent in the German Empire.

Birmingham sewage inquiry. 8vo. Birmingham and London, 1871.

This report contains a very thorough review of sewage purification as it existed in 1871, together with description and cuts of the pail system as used in Liverpool, Manchester, Rochdale, Birmingham, etc. The reader should remember, however, in reading the old reports, that many of the appliances which are illustrated have been improved within recent dates, and that the statements and illustrations can only be safely taken after one has obtained full knowledge of the subject and consequent power of selection.

Birmingham system of refuse disposal. Printed by order of the Birmingham Health Committee. Pamphlet; 8vo. Birmingham, 1893.

Blythe (A. Wynter). A manual of public health. 8vo. London, 1890.

Blythe (A. Wynter). Lectures on sanitary law. 8vo. London, 1893.

Contains twelve lectures on sanitary law delivered by the author at the College of Medicine as part of the usual course of instruction in sanitary science. Examples of by-laws relating to offensive trades, the law relating to adulteration, etc., are given in the appendix.

Boston main drainage works. 8vo. Boston, 1885.

Contains a full account of the construction of the main drainage works of the city of Boston, where, by reason of the nearness of the ocean, it was deemed preferable to dispose of the city sewage by discharge at high tide into the waters of the lower harbor.

Boulnois (H. Percy). Municipal and sanitary engineer's handbook. 8vo. London, 1893.

Brookman (F. W.). Utilization of town refuse at Rochdale. Reprint from the Journal of Society of Chemical Industry, April 30, 1895. London, 1895.

Buck. Treatise on hygiene and public health. 2 vols. 8vo. New York, 1879.

Burke (N. R.). A handbook of sewage utilization. 12mo. London, 1872.

Burn (R. Scott). Outlines of modern farming. 6th edition. 12mo. London, 1888.

Treats extensively among other subjects of the utilization of town sewage, irrigation, etc.

Callis (Robert W.). Lectures on sewers. Large 8vo. London, 1686.

Contains a series of lectures on sewers as delivered by the author, at Grace Inn, in August, 1622.

Chandler (C. F.). Report on the waters of the Hudson River, made to the water commissioners in the city of Albany. Paper; 8vo. New York, 1885.

The author, in 1885, takes the ground that the Hudson River was at the time a proper water supply for the city of Albany.

Chicago drainage and water supply. Preliminary report of a commission consisting of Rudolph Hering, Benezette Williams, and Samuel G. Artingstall. Paper; 8vo. Chicago, 1887.

Clippart (John H.). The principles and practice of land drainage. 3d edition. 12mo. *Cincinnati, 1888.*

Connecticut State Board of Health, 1884 to 1895, inclusive.

These reports contain a large amount of original work on pollution of water supplies and stream pollution generally as performed under the direction of the State board of health during the period included.

Cooley (Lyman E.). The Lakes and Gulf waterway as related to the Chicago sanitary problem. Paper; large 8vo. Chicago.

Coplin (W. M. L. and Bevan D.). A manual of practical hygiene. 8vo. Philadelphia, 1893.

Corfield (W. H.). The treatment and utilization of sewage. 3d edition, revised and enlarged by the author and Louis C. Parks. 8vo. London, 1887.

In this work the question of sewage utilization is examined at considerable length, and many useful conclusions are reached.

Crimp (W. Santo). Sewage disposal works. A guide to the construction of works for the prevention of pollution by sewage of rivers and estuaries. 1st edition. 8vo. London, 1890. 2d edition. London, 1894.

Contains descriptions up to date of a large number of the more important English sewage disposal works.

Dempsey (G. D.). Drainage of lands, towns, and buildings. Revised edition with large additions of recent practices in drainage engineering, by D. Kinnear Clark. 12mo. London, 1890.

Denton (J. Bailey). Sanitary works and sewage utilization. Paper; 8vo. London, 1869.

Denton (J. Bailey). Ten years' experience in works of intermittent downward filtration. 8vo. London.

Dibdin (W. J.). Report of experiments on the filtration of sewage effluent (from the London main drainage works) during the years 1892–1895, inclusive. Paper; 4to. London, 1895.

Engineering News. From 1892 to 1897, inclusive.

This journal contains more or less complete descriptions of the more interesting and important of the American works.

Engineering Record.

This journal has published a vast amount of information in regard to sewerage and sewage disposal both at home and abroad, and in 1889 published an index to matter pertaining to sewerage and sewage disposal which had appeared in Vols. V to XVII (December, 1881–June, 1888). There has also appeared a large amount of matter in the volumes from XVIII to XXXIV (the current volume).

Framingham, Massachusetts. System of sewage disposal. A report of a committee on drainage and sewerage and construction of a sewerage system. Statement in detail of cost of construction, plans, etc. 8vo. South Framingham, 1889.

Frankland (Percy and Mrs. Percy). Micro organisms in water, their significance, identification, and removal. 8vo. London, 1894.

Contains a statement of the relation of sewage-polluted water to disease.

French (Henry F.). Farm drainage. The principles, practice, and methods of draining land, etc. 12mo. New York, 1884.

Gerhardt (William Paul). The disposal of sewage at isolated country houses. Paper; 8vo. Providence, 1890.

Gerhardt (William Paul). The disposal of household wastes. 16mo. New York, 1890.

Gerhardt (William Paul). Sanitary engineering. Reprint from the Journal of the Franklin Institute, June–August, 1895. Paper; 8vo. Philadelphia, 1895.

Gray (Samuel M.). Proposed plan of a sewerage system for the disposal of the sewage of the city of Providence. Made by order of the city council of the city of Providence. Paper; 8vo. Providence, 1884.

Contains an account of a large number of sewage purification plants abroad as visited by Mr. Gray, together with recommendations for the partial purification of sewage of Providence by chemical treatment, followed by its discharge into tide water at a point where it could not become a nuisance along the adjacent beaches.

Gray (Samuel M.). Report showing location and size of the main intercepting sewers of the city of Providence and other information called for by the common council by resolutions of various dates in 1886. Paper; 8vo. Providence, 1886.

Griffiths (A. B.). A treatise on manures. 12mo. London, 1889.

Hall (William Hammond). Irrigation in California. 2 vols. 8vo. San Francisco, 1888, 1889. A report on irrigation in California as made by the State engineer.

A good text-book of American irrigation practice.

Harris (Joseph). Talks on manures. New and enlarged edition. 12mo. New York, 1888.

Hawkshaw (Sir John). Report as commissioner appointed to inquire as to the purification of the river Clyde. Paper; 4to. London, 1876.

Contains information as to the pollution of the river Clyde and its tributaries as they existed in 1876.

Hazen (Allen). Filtration of public water supplies. 8vo. New York, 1895.

Contains data as to quality of filtering material, construction of filters, etc.

Hazen (Allen). A practical plan for sand filtration as a means of securing a better water supply to the city of Philadelphia. Report to the Woman's Health Protective Association of Philadelphia. Paper; 8vo. 10 pp. 1896.

Health in country homes. The disposal of sewage. A series of articles reprinted from the New York Evening Post. Paper; 8vo.

Health of Towns Commission. This commission made two reports—the first, 1844, published in two 8vo. vols.; the second, in 1845, also in two 8vo. vols.

These two reports may be taken as the beginning of sanitary science in England and in the civilized world generally. These reports should be studied by any person wishing to study the whole subject of sewage utilization, by way of showing the magnitude of the evil which has been combatted and greatly mitigated since 1844.

Hill (John W.). Sewage disposal and water supply. A paper read before the Ohio Society of Surveyors and Civil Engineers, January 18, 1893. Reprinted for private circulation. Paper; 8vo.

Hill (John W.). Water supplies for eight cities in relation to typhoid-fever rates. An address before the Society of Municipal Improvements, Chicago, October 9, 1896. Paper; 12mo. 8 pp. Cincinnati, 1896.

Hill (John W.). The hygiene of water. A lecture delivered before the Teachers' Institute, Cincinnati, Ohio, September 11, 1896. Reprint from the Dietetic and Hygienic Gazette, October, 1896. Paper; small 4to. 10 pp. 1896.

Hutchinson (Christopher Clarke). On the disposal of sewage sludge. A paper read before the Society of Chemical Industry, February 4, 1884. Paper; 8vo. London, 1884.

Institution of Civil Engineers, Proceedings of. Vols. I–CXX. 8vo. London, 1838–1895.

Contain a large amount of information on river pollution and sewage deodorization, filtration, interception, irrigation, manure, precipitation, and sewage utilization generally. Detailed subject indexes have been issued from Vols. I–LVIII and from Vols. LIX–CXVIII, to which reference may be made for nearly every phase of the subject as discussed in England for the last forty to fifty years.

Johnson (Samuel W.). How crops feed. A treatise on the atmosphere and the soil as related to the growth of agricultural plants. 12mo. New York, 1890.

Johnson (Samuel W.). How crops grow. A treatise on the chemical composition, structure, and life of the plant. 12mo. New York, 1890.

Jones (Charles). Refuse destructors. 2d edition. 8vo. London, 1894.

Contains detailed information as to the result of garbage destruction by fire, as well as a paper on the utilization of town refuse for power production, by Thomas Tomlinson.

Kiersted (Wyncoop). A discussion of the prevailing theories and practice relating to sewage disposal. 12mo. New York, 1894.

The author believes that the rivers of the country are, on the whole, the natural place for disposing of sewage. In his view the natural forces of nitrification will purify the sewage in streams somewhat the same as on land.

nsett (C. T.). Nature's hygiene. A systematic manual of natural hygiene. 4th edition. 8vo. London, 1894.

epp (Charles Frederick). The sewage question. A general review of all systems and methods. 8vo. London, 1867.

Especially devoted to showing the superiority of the pneumatic system of Capt. Charles Liernur.

ichling (E.). Report on the proposed trunk sewer for the east side of the city of Rochester, New York. Pamphlet; 8vo. Rochester, 1889.

Contains a résumé of the various methods of sewage disposal and leans to the conclusion that for the conditions existing in Rochester a chemical treatment would be preferable.

itham (Baldwin). Sanitary engineering. A guide to the construction of works on sewerage and house drainage. 2d edition. 8vo. London, 1878.

This work was reprinted by Engineering News in 1877.

iwes (J. Parry). Reports of sewer air investigations, as made to the London County Council. Paper. London, 1894.

iffmann (Henry) and **Beam** (William). Examination of water for sanitation and domestic purposes. 12mo. Philadelphia, 1889.

ihmann (K. B.). Methods of practical hygiene. 2 vols. 8vo. London, 1893.

Translated from the German by W. Crooks.

ocal Government Board. Reports of the medical officer.

The reports of the medical officer of the privy council and local government board have contained for many years much information of interest and value relating to sewage purification and utilization. The student of the subject will find in these reports a vast amount of important matter. In the supplement to the report of the medical officer for 1891 may be found a study of enteric fever in the valley of the river Lee. In the supplement to the report for 1887 may be found papers giving full statistics of diarrhea and diphtheria in England. The first of these is a report by Dr. Ballard of the cause of the mortality from diarrhea which is observed principally in the summer seasons of the year in English communities, and the second is a statistical study by Dr. Longstaff on the geographical distribution of diphtheria in England and Wales. Dr. Ballard's statistical inquiry included the years from 1880 to 1888, while Dr. Longstaff's included the twenty-six years ending with 1880. The relation of these two diseases to sanitary conditions is set forth in many tables and diagrams with great clearness.

owcock (Richard Sidney). Experiments on the filtration of sewage. Excerpt from Proceedings Institution Civil Engineers, Vol. CXV. Paper; 8vo. London, 1893.

Iason (William P.). Water supply, chemical and sanitary. 8vo. New York, 1896.

Contains an excellent statement of drinking water as affected by sewage pollution in its relations to disease.

Iassachusetts Drainage Commission, Report of. 8vo. Boston, 1886.

This commission was authorized by the Massachusetts legislature to consider and report systems of drainage for the Mystic, Blackstone, and Charles river valleys. In 1885 a report of great value was submitted, in which questions of stream pollution and sewage disposal are discussed at length. The engineer of the commission, Elliot C. Clark, also submitted a report in which he gave the details of the problem of prevention of stream pollution and methods of sewage purification as applied to the river valleys named. One of the best of the early American reports.

Iassachusetts State Board of Health, Reports of.

This splendid series of sanitary reports now includes 27 volumes of annual reports and 2 special reports on the purification of sewage and water and on the examination of water supplies, besides special reports on the sewerage of the Mystic and Charles river valleys, and on the Metropolitan water supply of Boston, making 31 volumes in all. These are indispensable to any person who desires to study thoroughly the subject of sewage purification and utilization in the United States. Stream pollution, sewerage, and sewage disposal were first discussed in the Fourth Annual Report, and have been discussed in some phase ever since. In the Nineteenth Annual Report may be found the preliminary account of the special experiments *on intermittent filtration* carried out by the board at Lawrence.

Menzies (William). A treatise on the sanitary management and utilization of sewage. Large 8vo. London, 1865.

Metropolitan sewage discharge. Report of Royal Commissioners. 4 vols. of reports; minutes of evidence; appendixes, etc. 4to. London, 1884, 1885.

Presents every phase of question of disposal of sewage of London as it existed twelve years ago.

Metropolitan water supply, Report of Royal Commission on. 6 vols. General report; minutes of evidence; appendixes; index; plans, etc. 4to. London, 1893.

The most recent and extensive information as to pollution of streams and its effect on the water supply of the metropolis as applied to the rivers Thames and Lee, from which that supply is drawn.

Miles (Manly). Silos, ensilage, and silage. 12mo. New York, 1892.

A practical treatise on the construction of silos and the practical working of the process of ensilage.

Milwaukee, Wisconsin. Report of the Commission of Engineers on the collection and final disposal of the sewage and on the water supply of the city of Milwaukee. Paper; 8vo. Milwaukee, 1889.

Monson (Edward). The advantage of a separate system of drainage. Paper; 8vo. London, 1875.

Monson (Edward). Metropolitan sewage and what to do with it; a scheme of sewage disposal for the metropolis and the author's scheme of drainage for the lower Thames Valley, etc., the whole being a complete system of sewage clarification and purification. 8vo. London, 1883.

Moore (E. C. S.). Sanitary engineering. Occasional papers, Royal Engineers' Institute, being part of the professional papers of the Corps of Royal Engineers, Vol. XVII. 8vo. London, 1892.

Contains concise notes on various matters relating to sewerage and sewage disposal, as prepared specially for the use of the Royal Engineers.

Munroe (John M. H.). Composition and manurial value of filtered pressed sewage sludge. Reprint from the Journal of the Society of Chemical Industry, January 29, 1885. Manchester, 1885.

National Board of Health, Report for 1881.

This volume contains a report of the results of an examination made in 1880 of several sewerage works in Europe by Rudolph Hering. The information given, while extremely valuable in 1880, is of less interest now, by reason of being more recently covered. Mr. Hering gives a literature list which is still of value.

New Jersey State Board of Health, Vols. I–XVIII.

Contains information as to sewage-disposal works and stream pollution in New Jersey.

New York State Board of Health, Reports 1 to 16.

A large amount of information as to sewage disposal and stream pollution may be found in these reports, together with the detail plans of sewage-disposal plants thus far designed for towns in New York.

Palmberg (Albert). A treatise on public health and its applications in different European countries. Translated from the French. Edited by Arthur Newsholme. 8vo. London, 1893.

Contains a review of sanitary administration and legislation in the principal European countries.

Parks (Edward A.). A manual of practical hygiene. Edited by De Chaumont, with an appendix, giving the American practice in matters relating to hygiene, prepared by and under the supervision of Frederick N. Owen. 2 vols. Large 8vo. New York, 1884. From the last London edition.

Parks (Louis C.). Hygiene and public health. 12mo. Philadelphia, 1889.

Pennsylvania State Board of Health, Vols. I–X.

Contain many general discussions on stream pollution, sewerage, and sewage disposal.

Philadelphia Water Department, Annual Reports of the Chief Engineer, 1883 to 1886, inclusive.

These reports contain the results of an investigation as to the pollution of the water supply of the city of Philadelphia by sewage, together with the reports on additional supplies from unpolluted or nearly unpolluted sources, with methods of preventing pollution, etc. They may be referred to for much valuable information on the general question of pollution of streams and its attendant evils.

Philbrick (Edward S.). American sanitary engineering. 8vo. New York, 1881.

Philbrick (Edward S.). The disposal of sewage in suburban residences. Paper; 12mo. New York, 1883.

Platt (S. Sidney). Description of the sewage-disposal works at Roch Mills, Rochdale, England. Paper; 12mo. 4 pp. Rochdale, 1896.

Pollution of water supplies. Report of a committee of the American Public Health Association. Read at the annual meeting at Milwaukee, November 20–23, 1888. Paper; 8vo. Concord, 1889.

Provincial Board of Health of Ontario. Reports 1–4.

Contain information as to sewage-disposal works and stream pollution in the Dominion of Canada.

Rafter (George W.) and **Baker** (M. N.). Sewage disposal in the United States. 8vo. New York, 1893.

This work is dedicated to the health and prosperity of American cities, and contains descriptions and references to practically all the sewage-disposal works in the United States up to the summer of 1893.

Rauch (John H.). The sanitary problems of Chicago, past and present. Reprint from the Second Annual Report of the Illinois State Board of Health. Paper; 8vo.

Reeves (R. Harris). Sewer ventilation and sewage treatment. 12mo. London, 1889.

Contains description of special methods of chemical treatment and ventilation designed by the author.

Rivers Pollution Commission (first commission). 5 vols. 4to. London, 1866, 1867.

This commission made three reports. The first report deals generally with the best methods of preventing the pollution of rivers, with special reference to the conditions prevailing at that time on the river Thames. The second report deals with the river Lee. The third report deals with the rivers Ayr and Calder.

Rivers Pollution Commission (second commission). Report of the commissioners appointed in 1868 to inquire into the best means of preventing the pollution of rivers. 10 vols. 4to. London, 1870, 1871, 1872, 1874.

This commission made six reports in all. The first report, in two volumes, treats of the pollution of the basin of the rivers Mersey and Ribble and of the best means of preventing pollution therein. The second report is taken up with a description of the A, B, C process of treating sewage. The third report, in two volumes, discusses the pollution arising from the woolen manufacture and processes connected therewith. The fourth report treats of the pollution of the rivers of Scotland, and gives special consideration, among other subjects, to the pollution arising from paper-mill wastes, etc. The fifth report, in two volumes, treats of the pollution arising from mining operations and metal manufactures. The sixth report, in one volume, treats of the general subject of domestic water supply of Great Britain. In this report Dr. Edwin Franklin, the chemist member of the commission, has here worked out in detail the method of water analysis which he designated as the combustion method. A large amount of information about water supplies from cultivated and uncultivated areas and the contamination of water from manured and unmanured, cropped and uncropped land is given, the whole forming a vast body of sanitary information pertinent to present conditions.

Royal Sanitary Commission, Report of. Second report. 4to. London, 1871.

Contains a history of the English sanitary laws up to that date, with suggestions for new statute, etc.

Sanitary Institute, Transactions of. Vols. I–XII.

Contains many practical papers on sewerage and sewage disposal. 8vo. London.

Sewage disposal. Report of a committee appointed by the president of the local government board to inquire into the several modes of treating town sewage. Paper; 8vo. London, 1876.

Contains many details of sewerage work carried out in England to that date, with a large number of analyses of raw and effluent sewage at several sewage farms. In appendix No. 7 is given a list of a large number of patented processes for treating sewage and producing artificial manure therefrom, as taken out from between the years 1856 and 1875 inclusive. The report is also accompanied by a folio of plans, giving details of a number of the more interesting sewage-disposal works of that date.

Sewage disposal at Los Angeles, California. A report of the board of engineers upon the disposal of the sewage of Los Angeles city and its sewer system, presented to the mayor and common council of the city of Los Angeles, December 23, 1889. Paper; 8vo. 1889.

Contains the preliminary information as to sewage utilization at Los Angeles.

Sewage disposal at the London, Ontario, Asylum. Report of the Provincial Board of Health of Ontario. Paper; 8vo. Toronto, 1890.

Contains the report and detailed plans as submitted by Col. George E. Waring, jr.

Sewage disposal at the Rhode Island State institutions. Seventeenth Annual Report of the Board of State Charities and Corrections of Rhode Island, 1885. Paper; 8vo. Providence, 1886.

Contains the report and plans of Samuel M. Gray upon the disposal of the sewage of the State institutions at Cranston, Rhode Island.

Sewage disposal at the Rochester, Minnesota, State Hospital for the Insane. Sixth Biennial Report of the Trustees and Officers of the State Hospital, Rochester, Minnesota. 8vo. Minneapolis, 1890.

Contains report and detailed plans of the purification works at the Rochester Hospital.

Sewage farm competition of the Royal Agricultural Society of England. Report of judges appointed in the sewage farm competitions. Reprint from the journal of the Royal Agricultural Society. 8vo. London, 1880.

Sewage of the District of Columbia. Report by Rudolph Hering, Samuel M. Gray, and Frederick P. Stearns. Paper; 8vo. Washington, 1890.

Sewage of the Metropolis. Report from a select committee, ordered by the House of Commons to be printed July 14, 1864. Paper; 4to. London, 1864.

This committee was appointed specially to inquire into plans for dealing with the sewage of the metropolis (London) and other large towns with a view to its utilization in agriculture. The committee concluded that it is not only possible to utilize the sewage of towns, but that such an undertaking may be made to result in pecuniary advantage to the rate payers of the towns whose sewage is thus utilized. A large amount of evidence was taken by calling before the committee a number of the eminent engineers and English sanitarians of the day.

Sewage of Towns, Report from the select committee on. Two reports, in 2 volumes. First report ordered to be printed April 10, 1862, and the second report April 29, 1862. 4to. London, 1882.

Sewage of Towns Commission. Three reports. 8vo. London, 1858, 1861, 1865.

These reports contain the detail of elaborate investigations made by a royal commission appointed to inquire into the best mode of distributing the sewage of towns and applying it to beneficial and profitable uses. Elaborate cultivation and feeding experiments were pursued, extending over a period of several years, the results of which were presented in great detail in the second and third reports. In the appendix to the first report may be found an account of a visit made by a committee of the commission to Milan, Italy, for the purpose of examining the sewage utilization works which had been carried out at that place. This committee reported under date of December, 1857, that the experience of the irrigation around Milan adds a striking proof to those already obtained as to the value in agriculture of a command of pure water and of the immense increase of that value obtained by the addition of sewage combined with the higher temperature derived by the liquid in its passage through the town.

swage purification. Return of the particulars of the means adopted by local authorities for the purification or disposal of sewage, as made to the municipal authorities of the borough of Rochdale. 4to. 1872.

Contains details as to the population of towns, area of sewage farms or disposal areas, average rainfall of district, cost of land, number of years in operation, method of managing farm, information as to nuisances from effluent, etc. Also contains the particulars as to a large number of precipitating plants in operation in England in 1872.

swage treatment at the Barking and Crossness outfalls. Report by the engineer for the year 1894, ordered printed by the main drainage committee July 11, 1895. The same for the year 1893, ordered printed by the main drainage committee May 31, 1894. Five pamphlets in all. 4to. London.

swage utilization at the State Lunatic Asylum at Worcester, Massachusetts. Forty-seventh Annual Report of the Trustees, for the year ending September 30, 1879. Paper; 8vo. Boston, 1880.

Contains plans of the sewage utilization works as carried out in 1876 and as still in successful use at that institution.

iedd (J. Herbert). Report on sewerage of the city of Providence. Paper; 8vo. Providence, 1874.

ater (J. W.). Sewage treatment, purification, and utilization. 12mo. London, 1888.

Largely devoted to description and discussion of special processes of chemical purification. Contains a list of patents granted for sewage purification processes by chemical treatment, from 1886, inclusive. The total number of patents listed in that period is given at page 454, of which 11 have been taken out by the author of the work.

iow (F. Herbert). Report on sewer assessments. Paper; 8vo. Brockton, 1894.

aley (Cady) and **Pierson** (George S.). A separate system of sewerage, its theory and construction. Second edition; 8vo. New York, 1892.

Contains short chapter on sewage disposal.

orer (F. H.). Agriculture in some of its relations with chemistry. 2 vols. 8vo. New York, 1888.

Professor Storer's two volumes contain a survey of the whole field of scientific agriculture to date. As regards sewage utilization, following the English views of ten to fifteen years ago, he is disposed to say that it can not be made commercially successful. His work should be consulted by every person interested in the subject.

idy (C. Meynott). The treatment of sewage. 16mo. New York, 1887.

ieman (F.) and **Gartner** (A.). Die chemische und mikroskopisch-bakteriologische Untersuchung des Wassers. Paper; 8vo. Braunschweig, 1889.

ottenham Local Board of Health. A report on the disposal of sewage of the district, made by the Tottenham local board in May, 1870. 8vo. London, 1870.

urner (George). Report on the outbreak of enteric fever in the southeast of London. Made to the public health department May 18, 1892. Paper; 4to. London, 1892.

erwaltungs-bericht des Magistrats zu Berlin. Bericht der Deputation für die Verwaltung der Kanalisations Werke.

This is the annual official statement of the committee in charge of the sewerage system of the city. It contains full details as to the management of the sewage farms and the various scientific observations carried out there. It is from these statements that the tables given by Mr. Roechling in his paper on the sewerage works of Berlin have been prepared. The last statement at hand is that of the year from April 1, 1892, to March 31, 1893.

Vanklyn (J. Alfred). Water analysis. Practical treatise on the examination of potable water. 7th ed. 12 mo. New York, 1890.

Vardle (Thomas). Sewage treatment and disposal. 8 vo. London.

While this work has no date on the title-page, the preface is dated June 30, 1893. Its author, Thomas Wardle, esq., is a practicing English chemist of standing, who has himself perfected chemical processes of purification. The work contains much valuable information in regard to recent sewage disposal, and may be profitably consulted, although it is largely devoted to showing the superior value of Mr. Wardle's methods of purification over all others.

Waring (George E., jr.). **The sanitary drainage of houses and towns. 4th edition, revised and enlarged. 12mo. Boston, 1883.**

An excellent exposition of many of the fundamental questions.

Waring (George E., jr.). **Draining for profit and draining for health. 2d edition, 12mo. New York, 1883.**

Waring (George E., jr.). **Sewage disposal of isolated houses and large institutions. Reprint from the American Architect, March 12 to April 9, 1892.**

Waring (George E., jr.). **Modern methods of sewage disposal. 12mo. New York, 1894. A second edition has been published in 1896.**

Waring (George E., jr.). **Purification of sewage by forced aeration. Report of an experimental investigation of the value of the process of purifying sewage by means of artificially aerated bacterial filters. Paper; 8vo. Newport, Rhode Island, 1895.**

Waring (George E., jr.). **The disposal of a city's waste. Reprint from North American Review for July, 1895. Paper; 8vo.**

Waring (George E., jr.). **A report on the final disposition of the waste of New York by the department of street cleaning. Paper; 8vo. New York, 1896.**

Water supplies of Illinois and the pollution of its streams. A preliminary report to the Illinois State board of health, by John H. Rauch, secretary; with 2 appendixes: (1) Chemical investigations of the water supplies of Illinois, by Prof. J. H. Long; (2) The Illinois River Basin in its relations to sanitary engineering, by Lyman E. Cooley.

The beginning of very valuable studies, which unfortunately for sanitary science have not as yet been continued.

Wheeler (W. H.). **The drainage of fens and lowlands. 8vo. London, 1888.**

Wilcox (L.). **Irrigation farming. A handbook for the practical application of water in the production of crops. 12mo. New York, 1895.**

Wilson (George). **A handbook of hygiene and sanitary science. 5th edition. 12mo. Philadelphia, 1885.**

Worcester, Massachusetts. Reports of the city engineer in relation to the disposal of the sewage of the city of Worcester, made in 1889, in response to a request of the joint standing committee of the common council on sewers.

Contains an interesting account of a number of foreign sewage disposal works visited by the city engineer, as well as opinions of B. S. Brundell and James Mansergh in regard to the probable success of agricultural utilization at Worcester in comparison with sewage utilization in England, basing their opinions on the statements submitted as regards climate, etc. The sewage disposal works at Worcester were completed and put in operation in 1890. More recent reports, those of the superintendent of sewers, give detailed information as to cost of operation and degree of purification obtained. They may be consulted for a complete exhibit of the working from month to month of such a chemical-purification plant.

Worcester sewage and the Blackstone River. Reports on the pollution of the Blackstone River by the Worcester sewage, etc., with a project for the purification of the sewage of Worcester, by Col. George E. Waring, jr.

The French literature on sewage disposal is quite extensive, but is necessarily omitted because of lack of space. A number of new works and reports in English, which have appeared since the compilation of this list, are also omitted because of lack of space.

INDEX.

Page.

, N. Y., sewage disposal at..... 67
ɔe, Ohio, sewage disposal at..... 75
ıa, Pa., sewage disposal at..... 72-73
rst, Mass., sewage disposal at..... 42
ses of sewage..... 26-28, 44, 54, 63, 75
'er, Mass., sewage disposal at..... 56
ıa, sewage disposal in..... 82
ıd, Wis., typhoid fever at..... 39-40
:ic City, N. J., sewage disposal at.. 69-70
ıta, Me., sewage disposal at..... 41
ıa, N. Y., sewage disposal at..... 65
N. Y., sewage disposal at..... 65
, Canada, sewage disposal at..... 85-88
ıtone River, Massachusetts, pollu-
of..... 54
ı, Mass., water supply of..... 47-50
:ree, Mass., sewage disposal at..... 56
ters, N. Y., sewage disposal at..... 62
ɔwater, Mass., sewage disposal at.. 57
l, Conn., sewage disposal at..... 59-60
ton, Mass., sewage disposal at..... 42-45
lyn, N. Y., sewage disposal at..... 62-63
o, N. Y., typhoid fever at..... 39
rnia, sewage disposal in..... 82-85
a, sewage disposal in..... 85-89
ı, Ohio, sewage disposal at..... 75-76
ıl Falls, R. I., sewage disposal at... 57-58
auqua, N. Y., sewage disposal at... 63-64
;o, Ill., sewage disposal at.... 38, 40-41, 79
kill, N. Y., sewage disposal at..... 67
do, sewage disposal in 80-81
do Springs, Colo., sewage disposal
..... 80
rd, Mass., sewage disposal at..... 48, 56
rd, N. H., sewage disposal at..... 41-42
cticut, sewage disposal in.... 20-22, 59-62
nd, N. Y., sewage disposal at 67
' sewage-disposal works..... 63, 76
ɔn, R. I., sewage disposal at..... 59
use of sewage on various..... 45
ry, Conn., sewage disposal at..... 60
rs, Mass., sewage disposal at..... 56
ı, Minn., typhoid fever at..... 40
iffs, purification of..... 29-30
range, N. J., sewage disposal at... 70
aunton, Mass., sewage purification
..... 23-24
ıd, origin of sewage irrigation in.. 13
ıracter of sewage in..... 14-15
Fells, N. J., sewage disposal at..... 70
ckaway, N. Y., sewage disposal at. 64
zers, value of..... 37-38

Page.

Filter materials, experiments with..... 31-34
Flatbush, N. Y., sewage disposal at..... 67
Fonda, N. Y., sewage disposal at..... 68
Fort Plain, N. Y., sewage disposal at..... 67
Fostoria, Ohio, sewage disposal at..... 76
Foxboro, Mass., sewage disposal at..... 56
Fresno, Cal., sewage disposal at..... 82
Fulton, N. Y., sewage disposal at..... 68
Gardner, Mass., sewage disposal at..... 45-46
Genesee River, pollution of..... 71
Gowanda, N. Y., sewage disposal at..... 68
Great Lakes, pollution of..... 36-41
Greenfield, Mass., sewage disposal at..... 46
Hamilton, Canada, sewage disposal at... 88
Hastings, Nebr., sewage disposal at..... 80
Helena, Mont., sewage disposal at..... 81
Hemlock Lake, New York, pollution of.. 64-65
Herkimer, N. Y., sewage disposal at..... 68
Holly, N. Y., sewage disposal at..... 68
Hyde Park, Mass., sewage purification at. 23
Ilion, N. Y., sewage disposal at..... 68
Illinois, sewage disposal in..... 77-79
Intermittent filtration, purification by.. 34
Jamestown, N. Y., sewage disposal at.... 68
Kenton, Ohio, sewage purification at.... 25-26
Lake Calumet, Illinois, pollution of ... 77-79
Lake Cochituate, Massachusetts, pollu-
tion of..... 50-52
Lake Michigan, pollution of 38
Lake Wauremaug, Connecticut, pollution
of..... 60
Lawrence, Mass., experiments at..... 15, 30-35
Lawrenceville School, New Jersey, sew-
age disposal at..... 70
Leadville, Colo., sewage disposal at..... 80-81
Leicester, Mass., sewage disposal at..... 56
Lenox, Mass., sewage disposal at..... 46
Litchfield, Conn., sewage disposal at 60
London, Canada, sewage disposal at..... 88
Long Branch, N. J., sewage disposal at.. 70
Lorain, Ohio, typhoid fever at 39
Los Angeles, Cal., sewage disposal at.... 82-84
Lyons, N. Y., sewage disposal at..... 68
Maine, sewage disposal in 41
Manchester, Conn., sewage disposal at... 60
Manufacturing waste, purification of.... 27-35
Marlboro, Mass., sewage disposal at..... 46-47
Massachusetts, sewage disposal in..... 17-20,
23-24, 26, 42-57
Massachusetts board of health, experi-
ments of 15, 27-35
cited..... 17-18, 27, 34, 41, 44, 54-55

Page.
Massachusetts drainage commission cited 23
Medfield, Mass., sewage disposal at..... 48, 57
Menominee River, Wisconsin, pollution of 24-25
Menominee Valley, Wisconsin, sewage disposal in 79
Meriden, Conn., sewage disposal at 20-21, 61-62
Michigan, sewage disposal in..... 77
Millbury, Mass., sewage purification at.. 23
Milwaukee, Wis., sewage purification near..... 24-25, 38
Minnesota, sewage disposal in..... 79-80
Montana, sewage disposal in..... 81
Morris Plains Asylum, New Jersey, sewage disposal at..... 70-71
Mount Vernon, N. Y., sewage disposal at. 69
Mystic Valley, Massachusetts, sewage disposal in 48-49
Natick, Mass., sewage disposal at..... 50-51
Nebraska, sewage disposal in..... 80
Neponset River, Massachusetts, pollution of..... 18-20
New Britain, Conn., stream pollution at. 21
New Hampshire, sewage disposal in 41-42
New Jersey, sewage disposal in.... 15-17, 69-72
New Rochelle, N. Y., sewage disposal at.. 64
New York, sewage disposal in..... 62-69
Newell, F. H., letter of transmittal by... 9
Norfolk County, Mass., sewage disposal in..... 51
Norristown, Pa., sewage disposal at..... 73
North Brookfield, Mass., sewage disposal at..... 51
North Tarrytown, Mass., sewage disposal at..... 69
Norwood, Mass., experiments at..... 28
Oberlin, Ohio, sewage disposal at 76-77
Ohio, sewage disposal in 25-26, 75-77
Ohio board of health cited 25
Oneida, N.Y., sewage disposal at..... 69
Ontario Agricultural College, Canada, sewage disposal at..... 88-89
Paper-mill wastes, purification of..... 27
Paris, Tex., sewage disposal at..... 74-75
Pasadena, Cal., sewage disposal at..... 84-85
Passaic River, New Jersey, pollution of. 15-17
Pawtucket, R. I., sewage disposal at..... 58-59
Pegan Brook, Massachusetts, pollution of..... 50, 52
Pelham Manor, N. Y., sewage disposal at. 69
Pennsylvania, sewage disposal in..... 72-74
Phœnix, Ariz., sewage disposal at..... 82
Pittsfield, Mass., sewage disposal at..... 57
Plainfield, N. J., sewage disposal at..... 71
Poughkeepsie, N. Y., sewage disposal near..... 65-67
Princeton, N. J., sewage disposal at..... 71
Providence, R. I., sewage disposal at.. 22-23, 59
Publications relating to sewage disposal, list of..... 89-98
Pullman, Ill., sewage disposal at 77
Quinnipiac River, Connecticut, pollution of..... 20-22
Reading, Pa., sewage disposal at..... 73-74

Page.
Redding, Cal., sewage disposal at..... 85
Rhode Island, sewage disposal in.... 22-23, 57-59
Rivers Pollution Commission of England cited 14, 22, 29
Rochester, Minn., sewage disposal at..... 79
Rochester, N. Y., sewage disposal at..... 69
Rockingham County, N. H., sewage disposal in 42
St. Johns, Mich., sewage disposal at..... 77
Salt Lake City, Utah, sewage disposal at. 81-82
San Antonio, Tex., sewage disposal at.... 75
San Luis Obispo, Cal., sewage disposal at. 85
Sands, mechanical analyses of..... 31-34
Santa Rosa, Cal., sewage disposal at..... 85
Saylesville, R. I., sewage purification at. 23
Scioto River, pollution of..... 25-26
Sherborn Prison, Massachusetts, sewage disposal at 52
South Framingham, Mass., sewage disposal at 52-53
Spencer, Mass., sewage disposal at..... 57
Stockton, Cal., sewage disposal at..... 85
Stoneham, Mass., sewage purification at. 23
Stream pollution, discussion of..... 15-22
Sudbury River, Massachusetts, pollution of 19, 46-47
Summit, N. J., sewage disposal at..... 71-72
Tannery wastes, purification of 28-29
Taunton River, Massachusetts, pollution of..... 24
Temple, Tex., sewage disposal at..... 75
Tewksbury, Mass., sewage disposal at.... 57
Texas, sewage disposal in 74-75
Trinidad, Colo., sewage disposal at..... 81
Typhoid fever, occurrence of..... 38-41
United States, distribution of sewage-disposal plants in..... 11-13
Utah, sewage disposal in..... 81-82
Vassar College, New York, sewage disposal at 65-67
Wakefield, Mass., sewage disposal at..... 57
Wallingford, Conn., stream pollution at.. 21
Waltham, Mass., sewage disposal at..... 52
Waterbury, Conn., sewage disposal at 21-22, 62
Waterloo, Canada, sewage disposal at.... 89
Wayne, Pa., sewage disposal at..... 74
Wayne County, Mich., sewage disposal in. 77
Wellesley College, Massachusetts, sewage disposal at..... 53
West River, Rhode Island, pollution of.. 22-23
West Virginia, sewage disposal in 74
Westboro, Mass., sewage disposal at..... 53
Westfield, N. J., sewage disposal at..... 72
Weston, W. Va., sewage disposal at..... 74
Williams Bridge, N. Y., sewage disposal at 69
Winchester, Mass., sewage purification at 23
Windsor, Ontario, typhoid fever at..... 39
Wisconsin, sewage disposal in. 24-25, 79
Wisconsin board of health, cited..... 24
Wool-scouring wastes, purification of.... 27-28
Woonsocket, R. I., sewage disposal at.... 59
Worcester, Mass., sewage disposal at.... 53-56
Yalesville, Conn., stream pollution at.... 21

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 23

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

J. C. Branner

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

WATER-RIGHT PROBLEMS

OF THE

BIGHORN MOUNTAINS

BY

ELWOOD MEAD

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

CONTENTS.

	Page.
Letter of transmittal	9
Introduction	11
Number of appropriations and acreage reclaimed	12
Value of water rights	13
Board of control	14
Water divisions and districts	17
Appropriations in Division II	18
District 2, Crazy Woman Creek	18
Decree of 1889	18
Objections to the decree	21
Excessive amounts claimed	24
District 3	28
Flow of Clear Creek	28
Early rights and their adjudication	31
Diversion across divides	32
Prairie Dog Creek	35
Summary of rights to water	36
Allowance for loss of water in transmission	38
Ownership in ditch not necessary to the acquirement of a water right	40
District 4	43
District 5	45
Little Tongue River	49
Appropriations in Division III	50
District 6	50
Order establishing priorities and amounts of appropriations of water from No Wood River and its tributaries	50
Reservoirs	55
Administrative questions	58
Index	61

ILLUSTRATIONS.

	Page.
ATE I.—*A*, North side of Cloud Peak; *B*, Bighorn Mountains as seen from Buffalo, Wyoming	11
II.—Piney Creek below outlet of Cloud Peak Lake	32
III.—Reservoir sites on Piney Creek: *A*, At Elk Lake; *B*, At Mead Lake	34
IV.—North slope of Massacre Hill, illustrating erosion in ditches from Piney Creek	36
V.—Timber flume in the canyon of Tongue River	44
VI.—Beginning of dam at north end of Dome Lake reservoir	54
VII.—Outlet of Kearney Lake reservoir site, on North Piney Creek	58
G. 1. Location of water districts in portions of Divisions II and III	17
2. Irrigation systems on Clear Creek and tributaries	29
3. Discharge of Clear Creek, near Buffalo, Wyoming, for 1894, 1896, and 1897	31
4. Ditches diverting water from Piney Creek across Massacre Hill divide to Prairie Dog Creek	35
5. Location of ditches on tributaries of Tongue River, Division II, district 4	44
6. Location of ditches on North Fork of Tongue River, Division II, district 5	46
7. Dome Lake reservoir	56
8. Lakes at head of Piney and Goose creeks	57

LETTER OF TRANSMITTAL.

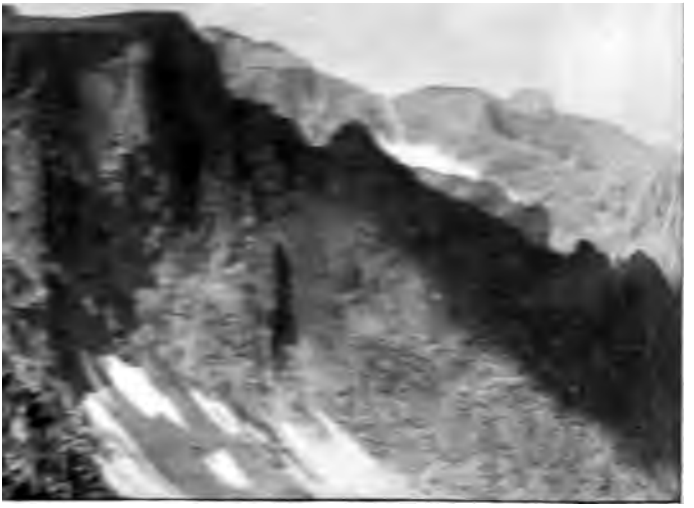
DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, October 28, 1898.

SIR: I have the honor to transmit herewith the manuscript for a paper entitled Water-Right Problems of the Bighorn Mountains, by Prof. Elwood Mead, State engineer of Wyoming. This paper calls particular attention to the complications arising in the distribution of water as a result of the haphazard construction of small irrigation ditches. The problems encountered in this region are typical of those which are to be met in nearly every State of the West, or which will be met in the near future. The difficulties found in the area described are accentuated by the diversion of water from one stream across divides into other natural drainage lines, thus connecting and greatly complicating the water rights of one individual with those of others. The calling of public attention to these conditions will go far toward arousing enlightened sentiments in behalf of more comprehensive laws in other States and should bring needed support to the efforts of officials in their attempts to distribute equitably the waters of the flowing streams.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.



WATER-RIGHT PROBLEMS OF THE BIGHORN MOUNTAINS.

By ELWOOD MEAD.

INTRODUCTION.

The Bighorn Range is the most impressive feature in the landscape of northern Wyoming, and its scenic beauties are made the more attractive by the striking contrast they afford to the surrounding region. The traveler from the East or the South finds the clear, cold, perennial streams which flow from its snow-clad summits a grateful relief from the alkaline pools or dry sand channels which have to be largely relied on when journeying toward them. The lofty and rugged pinnacles of Cloud Peak (shown in Pl. I, *A*) are made all the more commanding because the bad-land hills, which form the summit of the Cheyenne watershed, are so commonplace and unattractive. West of the range is the Bighorn Basin, the lowest and most arid section of the State, where the dusty roads and gray stretches of sagebrush make a traveler look with longing at the snow-crowned, forest-clad slopes of the mountain barrier to the east.

In the arid States mountains, not plains, control agricultural development, and this isolated barrier, which rises near the middle of the northern boundary and extends southeast nearly to the center of the State, has done more to people the northern half of Wyoming than all other influences combined. Without it the section, now gridironed with ditches and supporting about one-fourth the population of the State, would be an open range supporting a few migratory flocks of sheep and herds of cattle. The latent wealth of the few hundred square miles of rocky, barren, and almost unoccupied land surface, over 8,000 feet above sea level, is far greater than that of any equal area in the valley below. This region gathers and stores the moisture which alone gives value to the land below. The perennial streams thereby created have, in their diversion and use, given employment to more people, attracted the investment of more capital, and created more problems to tax the wisdom of the lawmakers than has all the unimproved unirrigated farming land in the State.

The ease and cheapness with which the streams can be diverted, the fertility of the soil, and the exceptional value of the surrounding grazing land have made this section an attractive location for th-

seeker of limited means. It is a region of small ditches and individual appropriators of water. On the eastern slope appropriation began before there were any laws governing the recording or establishment of rights thereto, while the isolation of the early settlers made them disregard or give little heed to water-right problems until the needs of users began to exceed the available supply; since then the multitude of rights, resulting from the haphazard location of ditches, has made a satisfactory settlement of many of these problems unusually difficult. Before discussing these problems a brief review will be given of the progress made in the twenty years which have elapsed since Manlius Redmond, a settler on Clear Creek, built the first ditch and became the pioneer farmer of the northern half of the State.

At that time the nearest railway station in the State was Rock Creek, on the Union Pacific, nearly 300 miles away. There was not a flour mill in the State, and the long, hazardous, and expensive trip necessary to secure the plow which turned the first furrow and the food for the men who worked on it added greatly to its cost. Reclaiming the desert under such conditions had many drawbacks, and the earlier ditches were of the crudest and simplest character. There was no attempt at any comprehensive projects. Each irrigator was a law to himself. Cooperative ditches or canals were rare exceptions. This explains the large number of ditches of record, and is the cause of many of the obstacles to the satisfactory division of water which confront those charged with this duty.

NUMBER OF APPROPRIATIONS AND ACREAGE RECLAIMED.

The records of the State engineer's office show that there are 1,051 adjudicated rights to the water flowing out of these mountains. There are also 269 claims which have been recorded and approved, but not adjudicated. In addition there are probably 100 ditches using water for which no claim has been filed. The adjudicated rights describe 139,478 acres of land to which these rights are attached in the orders of the board of control establishing them, and there are appropriations from Crazy Woman Creek aggregating 925.25 second-feet where the land irrigated is not described. Four flour mills have rights to water for power purposes aggregating 324.50 second-feet, and the several cities and towns of this region have adjudicated rights for power and domestic purposes amounting altogether to 80.6 second-feet.

Along the northern end of the range some promising placer fields have been discovered, and rights to 137.6 second-feet have been acquired with which to work them. The total volume of the adjudicated rights (3,455.4 second-feet) is in excess of the low-water discharge of the streams against which they are held, but is below the maximum discharge, so that the securing of a stable and adequate water supply for many of the irrigators holding adjudicated rights is a question of

storage. While all of the rights for irrigation, established by the board of control, are based on State surveys of the ditches and on sworn statements of the acreage irrigated by the appropriators, there is no question that these statements are somewhat in excess of the truth. Irrigation and reclamation are elastic terms. Spreading water over land to increase the growth of grass for pasturage purposes is irrigation and is so held by the board of control, but it is not the complete reclamation that is accomplished by cultivation. The tendency to augment the acreage actually watered is so common as to lead to the belief that it is unconscious. In the absence of exaggeration so gross as to constitute an injustice to other users, such a claim usually passes without protest unless it includes land which the State engineer's surveys show to be not susceptible of irrigation. The total acreage irrigated is, therefore, below the figures before given. A deduction of 10 per cent for exaggerated acreages and for areas which have only a surplus or flood-water supply would not be excessive, and would leave 125,000 acres as the actual area irrigated under rights established by the board of control. This is more than half the acreage given as irrigated in the entire State by the Tenth Census.

It is impossible to speak with equal certainty regarding the acreage irrigated under appropriations approved by the State engineer's office, but which have not yet been established by order of the board of control. In making a summary of these rights from the records of the engineer's office, all canceled permits, or those which have not complied with the requirements of the State law, have been excluded. This disposes of all the speculative filings which were made prior to 1894, during the time that no fee was required for the examination and record of applications to appropriate water. Since that time the stringent regulations governing the preparation of these filings have made applications so expensive that the tendency to make these for speculative purposes has almost disappeared. While the applications for permits recently approved do not represent actual utilization, they do represent projects based on reliable surveys, which are being carried out by people who are in earnest and who have sufficient investment in the work to make its completion probable. This is more nearly true of the appropriation of water from the streams of this region than of any other section of the State, as there are no large projects. These recorded and approved permits describe 214,000 acres of land for which irrigation appropriations are claimed, and 34 rights to water for power and domestic purposes.

VALUE OF WATER RIGHTS.

The value of the rights to the water which fills these ditches and fertilizes the land under them is an interesting subject for consideration. That they give to both ditches and land whatever value they have is unquestioned, because an empty ditch and an arid plain are alike

worthless. But while the right to water gives to the land whatever value it has, it does not make all values alike. Cost of maintaining ditches, ease of distributing water, fertility of the soil when irrigated, and the value and extent of the contiguous range are elements which have their influence. The fact that transfers of these rights have not thus far been recognized by the irrigation authorities of Wyoming has prevented any transactions of this character, so that we have to go to surrounding States for statistics on this subject. A recent sale of a right to 78 second-feet in one of the streams of northern Colorado is perhaps the best illustration of the present value of this form of property. In this sale nothing was sold but the right to take water from the stream. The ditches and lands by which it was conducted remained in the possession of the original holders. The right was moved several miles from its original location and required another ditch for the diversion of the water, and yet sold for a little more than $250 per second-foot. Its value was also lessened by the fact that the purchasers knew that the sale would be contested and that the expense of litigation to establish the legality of the transfer would have to be paid by them. The price paid, therefore, was far below what the property would have brought had the sellers been able to give an unquestioned title to the property, and is not, I think, above the value of an unquestioned right in the Bighorn Mountains. If this is true, the value of the adjudicated rights to the water of this range is already over $1,000,000. That this is destined to be greatly augmented in the near future is beyond question. The commercial importance of the water rights, therefore, makes their legality and the measures devised for their enforcement a subject of great economic interest.

BOARD OF CONTROL.

Under the constitution of Wyoming the board of control is intrusted with the supervision and control of all public waters of the State and of their appropriation, diversion, and distribution. Its administrative powers are therefore very broad. It has not only to determine rights to streams, but to enforce those rights afterwards. It has five members. These are the State engineer, who is president, and the four division superintendents. At the outset the board endeavored to outline clearly and definitely its views of what was necessary to complete or perfect a title to water, and so to simplify the methods of taking proof as to enable all water users to know what was required and to prepare and submit it at the least possible outlay. In pursuance of these views the board adopted and published the following governing principles:

1. In order to constitute a valid appropriation the water must have been used for some beneficial purpose, and in the case of appropriation for irrigation the water must have actually been applied to the land.

2. The amount of an appropriation is governed by the volume used and by the requirements of this use—in the case of appropriations for irrigation, by the needs of the land reclaimed.

3. Where reasonable diligence is shown in the construction of ditches and utilizing water, the appropriation dates from the beginning of work on the ditch, the survey to be considered as part of such work; where reasonable diligence is not shown, the appropriation dates from the utilization of the water.

4. Priority of appropriation gives priority of right, except in the case of appropriations made between 1888 and 1891, during which time the law made appropriation for domestic use a preferred priority.

5. Appropriations for irrigation are restricted to 1 cubic foot per second for each 70 acres irrigated. While this does not necessarily apply to lands reclaimed before the enactment of the State law, no appropriation for a larger amount has been made, because in all cases so far considered this volume has appeared ample.

6. Transfers of rights to water made in advance of any adjudication, either by the courts or by the board, are not recognized, the reason being that parties had not such ownership as would enable them to give valid title to the water sold. No transfers involving changes in location or character of use have been recognized.

As the operation of these principles is uniform, and as the facts on which any appropriation rests are in all cases the same, it was possible to prepare a blank form of proof for the guidance of irrigators, and in that way to simplify both the labor of preparing proofs and of the board in their subsequent examination. The following is the form adopted:

PROOF OF THE APPROPRIATION OF WATER.

From ———, Division No. ——.

State your name.

1. Post-office.

2. State the use to which the water has been applied.

3. State the means of diversion employed.

4. If through a ditch, state its name.

5. State (*a*) the date of the survey of the ditch or other distributing works through which the water claimed is diverted; (*b*) the date when the construction of such ditch was begun and when completed.

6. If any enlargements were made, state the date when begun and the date when completed.

7. State the dimensions of the ditch as originally constructed and as enlarged, and if measured by the county surveyor under the provisions of the act of 1886, give the results of such measurements.

8. State the name of person, association of persons, or corporation who built the ditch or canal, and the name or names of its present owners.

9. If water is claimed for irrigation, give the legal subdivisions of land owned or controlled by you for which an appropriation is claimed.

10. State the nature of your title to the above-described land: and if not owned by you, *give the name of the owner* and the nature of the possessory right which *you exercise.*

11. State the year when water was first used for irrigation or other beneficial purposes, and by whom.

12. If for irrigation, give the number of acres watered the first year, giving the legal subdivisions on which used and, as near as may be, the acres irrigated in each subdivision.

13. State the number of acres watered each subsequent year, and give the legal subdivisions on which the water was used and, as near as may be, the acres irrigated in each legal subdivision.

14. State the number of acres irrigated from said ditch in 189—, and give the legal subdivisions on which water was used and, as near as may be, the acres irrigated in each subdivision.

15. State the acreage said ditch is capable of watering, give the legal subdivisions of land which it can be made to irrigate, and state who owns said land.

16. State your proportionate interest in said ditch.

(16) The plat prepared by the State engineer is hereby accepted as showing correctly the location of the ditch and the land which can be irrigated therefrom.[1]

17. When does your irrigation season begin and when does it end?

18. If water is used for other purposes than irrigation, state the nature of such use and the time when such use began.

19. How much water is required for such purpose?

20. During what months is the water used?

21. Have you or any other claimant of said appropriation filed a claim to water in the office of the county clerk? If so, give date of filing and the name of the party or parties interested in said claim.

22. Have you had sufficient water each year since the use for which an appropriation is claimed began?

23. If not, state the years of scarcity, the months when the supply was insufficient, and the reason of such scarcity.

I hereby certify that the foregoing affidavit was read to the affiant in my presence before he signed his name thereto; that said affiant is to me personally known (or has been satisfactorily identified before me by ——— ———), and that I verily believe him to be a credible person and the person he represents himself to be; and that this affidavit was subscribed and sworn to before me at ———, on this —— day of ———, 189–.

———— ————,
Division Superintendent.

DEAR SIR: Your attention is called to the inclosed proof of appropriation. It is the intention of the law and also of the board of control to make the adjudication of rights to the use of water as inexpensive to the appropriator as possible. Consequently you are requested to cooperate with the division superintendent to whom you submit your testimony to the extent that you prepare yourself to answer all questions readily and accurately before the day set for submitting said testimony.

Your special attention is called to those questions whose answers indicate the date of your appropriation and use of the water, and to the acreage of land irrigated and description thereof.

By complying with this request you will not only facilitate the taking of testimony, but will insure the correct establishment of the respective rights and possibly prevent expensive and aggravating litigation.

Yours truly,

———— ————,
Division Superintendent.

[1] In case there is objection to the official plat, the parties objecting must, when giving proof, file a written statement of the reasons therefor, and must within thirty days file with the division superintendent a map of said ditch and irrigated lands, with affidavit of surveyor, giving of survey, attached thereto.

WATER DIVISIONS AND DISTRICTS.

In order that each of the more than 1,000 holders of water rights shall obtain his share of the common supply there must be some officer to see that the rights are enforced and protected in times of

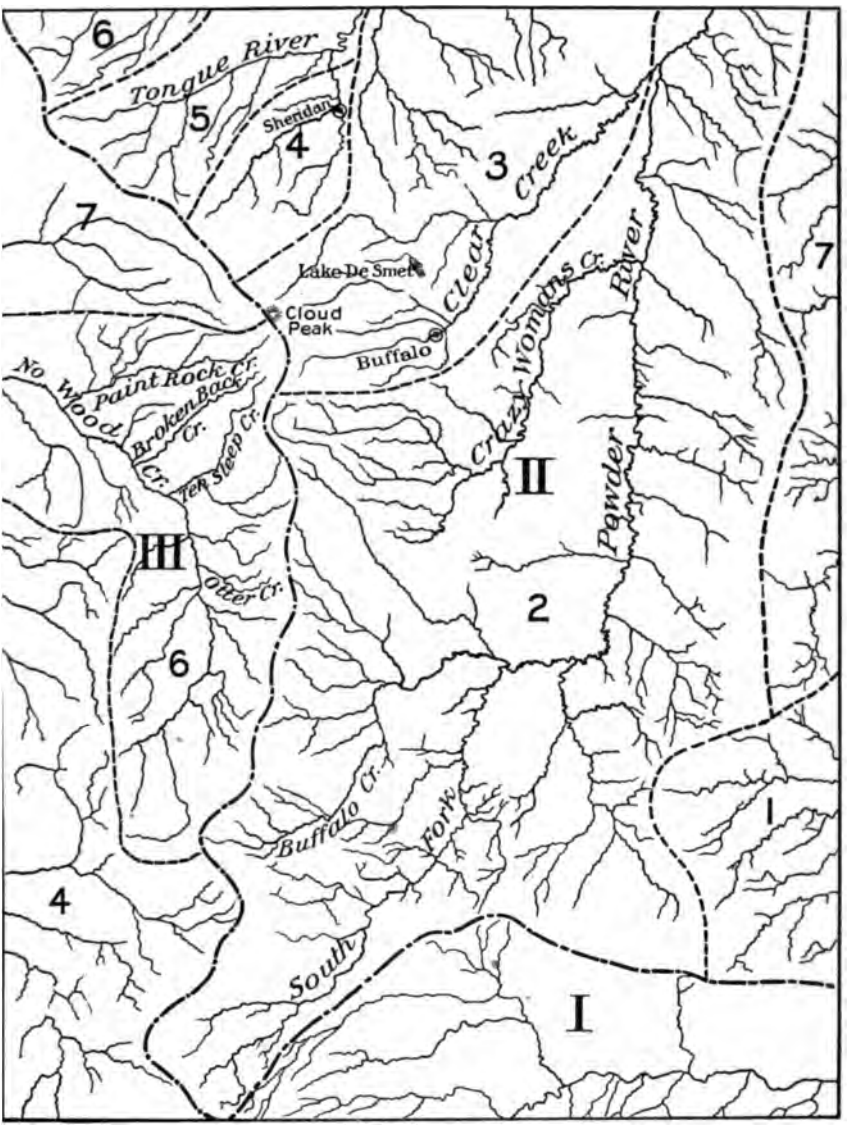


FIG. 1.—Location of water districts in portions of Divisions II and III.

scarcity. For convenience, and in order to make the work more effective, the entire State has been divided into four parts, called *water divisions*, and these divisions have been subdivided into dis-

Dimensions of ditches taking water from Crazy Woman Creek.

Priority number.	Name of ditch.	Average dimensions.			
		Top width.	Bottom width.	Depth.	Grade per mile.
		Feet.	*Feet.*	*Feet.*	*Feet.*
1	Billy Creek	7	5	1	7.2
2	John R. Smith	7.5	5	2.75	12
3	North Fork	.....	4	1.5	24
4	Harris No. 1	6	4	1.50	6.25
5	Kingsbury No. 1	2	1	1	73.95
6	Halt	5.5	3.5	1	16.5
7	Red Bark	9.5	7.5	1.2	12.7
8	Moreton	6.2	4	1.1	19
9	Cook	10	6	1.8	8
10	North Fork of Crazy Woman	7	4.5	2	4.5
11	Muddy Creek	6	4	1	5
12	Px	7.6	5	1.3	5
13	Kingsbury No. 4	2	1	1	29.94
14	Kingsbury No. 3	2	1	1	29½
15	Kingsbury No. 2	2	1	1	71½
16	Kennedy	*a* 7.1	.....	2.3	5.3
17	Dick	8.5	5	1.75	10
18	Burton, Kelly Creek	*b* 1.5	.....	1	15
19	Blue Gap	7.5	5.5	1	23
19½	Teddy Miller	13.75	7	2.25	12
20	Fort Collins	20	10	3.7	3.17
20½	North Fork (*c*)	8.5	5.5	1.5	24
21	Devoe No. 1	14	10	2	8
22	Devoe No. 2	10	8	2.5	5.28
23	Canton, Kelly Creek	5	3	1.5	5.28
24	Thompson Brothers	6.5	3.5	1	20
25	Thompson & Matthews	*d* 9	5	1	(*e*)
25½	Moreton enlargement	.....	.....	1.9	5.28
26	Mitchell & Long	.....	6	1.5	4
26½	Kennedy enlargement	11.7	7.1	2.3	24
27	Harris No. 2	.....	3	1	6.28

a Average width as originally constructed.
b Average width.
c Enlarged by Covington and Mays.
d Average width as enlarged.
e Grade 4 feet to the mile part of the way and 15 feet for the remainder.

OBJECTIONS TO THE DECREE.

The water described in this decree would suffice for the irrigation of between 50,000 and 75,000 acres of land even under the wasteful methods which prevail. The property disposed of was of great value. If a second-foot of water is worth $250, rights to 900 second-feet are of enough importance to warrant an inquiry into the methods by which they were acquired. Nothing is more indispensable, if we are to avoid disastrous controversies, than to have the acts by which a right to water is secured clearly defined and the limitations on that right definitely prescribed by law. Every user of water wishes to know how the title thereto is to be secured, while everyone interested in the growth and success of irrigated agriculture wishes to be certain that the commodity on which its success depends shall not be wastefully or fraudulently transferred to private or speculative owners. The results of this adjudication have apparently not answered either of the questions clearly or to the satisfaction of those directly concerned.

Controversies over the legality of these rights and over the discretionary authority of the water commissioner in enforcing the decree began the first season of its operation and still continue. That all should have agreed was out of the question. The Territorial law was both indefinite and contradictory in its provisions. Those claiming rights were not agreed either as to the meaning of the law or in their views of an irrigator's needs. Nor was the method of taking proof calculated to disclose the facts on which a just or satisfactory settlement of these rights must rest. There was no measure of the flow of the stream. There was no survey of the ditches, of the lands irrigated, or of the lands which could be irrigated. There was no proof of the actual volume of water diverted or of the needs of the land for which its use was claimed. All that was required in this disposal of public property was for each individual claiming a right therein to file proof of the building of a ditch and his estimate of the volume it would divert. The first objection to this was the lack of knowledge of the claimants themselves. Not one of them knew accurately the carrying capacity of his ditch or the volume needed to reclaim an acre of land. To all the expression "cubic foot per second" was an almost meaningless term. It would have meant just as much if the volume had been expressed in liters. Granting that the desire of all was to claim only what they were entitled to, their success in doing this would be wholly accidental. Nor was the submission of these statements to a judge or a commissioner having no greater practical knowledge than the irrigators themselves a wise provision. The questions to be determined were physical, not legal, and without some knowledge of either the volume which the stream carried or the water used on an acre of land these officers were in no way prepared to pass on the reasonableness of the claims.

seeker of limited means. It is a region of small ditches and individual appropriators of water. On the eastern slope appropriation began before there were any laws governing the recording or establishment of rights thereto, while the isolation of the early settlers made them disregard or give little heed to water-right problems until the needs of users began to exceed the available supply; since then the multitude of rights, resulting from the haphazard location of ditches, has made a satisfactory settlement of many of these problems unusually difficult. Before discussing these problems a brief review will be given of the progress made in the twenty years which have elapsed since Manlius Redmond, a settler on Clear Creek, built the first ditch and became the pioneer farmer of the northern half of the State.

At that time the nearest railway station in the State was Rock Creek, on the Union Pacific, nearly 300 miles away. There was not a flour mill in the State, and the long, hazardous, and expensive trip necessary to secure the plow which turned the first furrow and the food for the men who worked on it added greatly to its cost. Reclaiming the desert under such conditions had many drawbacks, and the earlier ditches were of the crudest and simplest character. There was no attempt at any comprehensive projects. Each irrigator was a law to himself. Cooperative ditches or canals were rare exceptions. This explains the large number of ditches of record, and is the cause of many of the obstacles to the satisfactory division of water which confront those charged with this duty.

NUMBER OF APPROPRIATIONS AND ACREAGE RECLAIMED.

The records of the State engineer's office show that there are 1,051 adjudicated rights to the water flowing out of these mountains. There are also 269 claims which have been recorded and approved, but not adjudicated. In addition there are probably 100 ditches using water for which no claim has been filed. The adjudicated rights describe 139,478 acres of land to which these rights are attached in the orders of the board of control establishing them, and there are appropriations from Crazy Woman Creek aggregating 925.25 second-feet where the land irrigated is not described. Four flour mills have rights to water for power purposes aggregating 324.50 second-feet, and the several cities and towns of this region have adjudicated rights for power and domestic purposes amounting altogether to 80.6 second-feet.

Along the northern end of the range some promising placer fields have been discovered, and rights to 137.6 second-feet have been acquired with which to work them. The total volume of the adjudicated rights (3,455.4 second-feet) is in excess of the low-water discharge of the streams against which they are held, but is below the maximum discharge, so that the securing of a stable and adequate water supply for many of the irrigators holding adjudicated rights is a question of

storage. While all of the rights for irrigation, established by the board of control, are based on State surveys of the ditches and on sworn statements of the acreage irrigated by the appropriators, there is no question that these statements are somewhat in excess of the truth. Irrigation and reclamation are elastic terms. Spreading water over land to increase the growth of grass for pasturage purposes is irrigation and is so held by the board of control, but it is not the complete reclamation that is accomplished by cultivation. The tendency to augment the acreage actually watered is so common as to lead to the belief that it is unconscious. In the absence of exaggeration so gross as to constitute an injustice to other users, such a claim usually passes without protest unless it includes land which the State engineer's surveys show to be not susceptible of irrigation. The total acreage irrigated is, therefore, below the figures before given. A deduction of 10 per cent for exaggerated acreages and for areas which have only a surplus or flood-water supply would not be excessive, and would leave 125,000 acres as the actual area irrigated under rights established by the board of control. This is more than half the acreage given as irrigated in the entire State by the Tenth Census.

It is impossible to speak with equal certainty regarding the acreage irrigated under appropriations approved by the State engineer's office, but which have not yet been established by order of the board of control. In making a summary of these rights from the records of the engineer's office, all canceled permits, or those which have not complied with the requirements of the State law, have been excluded. This disposes of all the speculative filings which were made prior to 1894, during the time that no fee was required for the examination and record of applications to appropriate water. Since that time the stringent regulations governing the preparation of these filings have made applications so expensive that the tendency to make these for speculative purposes has almost disappeared. While the applications for permits recently approved do not represent actual utilization, they do represent projects based on reliable surveys, which are being carried out by people who are in earnest and who have sufficient investment in the work to make its completion probable. This is more nearly true of the appropriation of water from the streams of this region than of any other section of the State, as there are no large projects. These recorded and approved permits describe 214,000 acres of land for which irrigation appropriations are claimed, and 34 rights to water for power and domestic purposes.

VALUE OF WATER RIGHTS.

The value of the rights to the water which fills these ditches and fertilizes the land under them is an interesting subject for consideration. That they give to both ditches and land whatever value they have is unquestioned, because an empty ditch and an arid plain are alike

While rights may have been acquired through the building of these ditches, the volumes decreed were manifestly not based on their capacity. Appropriator No. 1, after building a ditch to carry 16 feet, was given only a fraction over half that amount, while in contrast thereto the third appropriator was given 11 feet more than his ditch would divert. The seventh appropriator was given 2 second-feet for each foot the ditch could carry, while the seventeenth was cut down to less than half what had been or could be taken from the stream. Priority No. 18 was favored with a double allowance, while No. 20½ got only half a loaf. Some of the appropriations agree approximately with the volume which the ditch could divert, but the number which are either far above or far below shows that this agreement was accidental and not based on any investigation of this question.

EXCESSIVE AMOUNTS CLAIMED.

Nearly all recent laws and decisions are agreed that simply diverting water does not of itself complete a right to it; that in addition the water so diverted must be applied to some beneficial use. If this test be applied to appropriations for irrigation, the actual acreage reclaimed might properly be taken as the measure by which the volume appropriated should be determined. All of these appropriations are for this purpose, and all except one of them, No. 16, states the acreage to be irrigated, and in the excepted case it is restricted to the land owned by the claimants. A casual inspection of the number of acres irrigated and the amount decreed shows that the need of the land had no weight in fixing these rights. While the measurements of the water used in irrigating an acre of land have not as yet taken the range or embraced all the conditions needed to fix the absolute volume to be used on that area, enough have been made to show the approximate limits of such use. In a series of tests begun by the State engineer and continued by the State University of Wyoming there was not a single instance in which the duty of a second-foot fell below 80 acres. As these tests were made in widely separated localities, embraced a wide range of crops, and were continued for five years, this may fairly be taken as a limit of the actual needs of irrigation, and the State legislature has, since this decree was rendered, made any appropriation in excess of 1 second-foot to 70 acres illegal.

The land along Crazy Woman Creek can be irrigated with less water than is required in some of the localities where the volume used was measured, and there is no reason to question the statement that a duty of 1 second-foot to 80 acres is below rather than above the actual practice along the stream. But whether this be true or not, it is certain that the need of 1 acre is approximately the same as that of every other acre. There is so little variation in the soil and in the crops grown, that sufficient water for one 80-acre tract is enough for any other equal area along the same stream.

This fact could not have been considered in fixing these rights. In the table given, one column gives the acres to be irrigated; in the next column are the acres for which 1 second-foot of water was decreed. It will be seen that the first appropriator is given 1 second-foot for each 18 acres of his land, or at least four times the volume needed, while the fourteenth appropriator is required to spread a second-foot over 133 acres. When the owner of priority No. 20 found he was allowed only 1 second-foot for 100 acres, while his neighbor, owning the twenty-seventh, was given the same volume for 10 acres, he could not be blamed for objecting to the discrimination.

All of these awards are for a continuous flow for the entire year. In practice, however, the use of water in irrigation extends over a period of only three to four months. Assuming one hundred days to be the time of actual need, the irrigator under priority No. 1 is allowed water enough to cover his land to a depth of 10.8 feet, while the irrigator under priority No. 14 can cover his land with only 1.5 feet. The irrigator under priority No. 24 can, if he desires, submerge his farm under nearly 22 feet of water, while the appropriator under priority No. 13 can cover his with less than that many inches.

The excessive volumes of these appropriations make it manifest that the necessities of the land irrigated had no more to do with determining the volume decreed than had the size of the ditches. It is necessary to look further, therefore, to learn the reason for making some rights exceed the capacity of ditches and others fall below, and for allowing some appropriators more water for 80 acres than was given others for 800.

In 1886 a law was passed requiring parties claiming appropriations of water to file in the county clerk's office a statement of their claim. This law required that the statement should contain a description of the ditch, the name of the stream, and the number of acres lying under the ditch and irrigated or proposed to be irrigated. In the same law was a provision for a similar statement of claim for proposed ditches, which required any party intending to appropriate water to file a similar statement with the county clerk. It contained the same provisions as the section relating to existing ditches, with several additional ones relating to protection of existing rights and fixing the time for beginning of work, together with the following ambiguous and misleading clause:

> And from the time of filing any such statement water sufficient to fill such ditch or ditches and to subserve the use or uses aforesaid, if a lawful and just use, *shall be deemed and adjudged to be appropriated.*

Those who know anything about the difficulties which beset engineers in the measurement of water can realize how difficult, if not impossible, it was for farmers having no knowledge of hydraulics to make a correct estimate of the volume they had actually diverted and used, and that it was impossible for them to make an accurate

claim for the volume they proposed to use. A law which would give to each irrigator all he claimed would be an absurdity, if every claim was known to be limited to the claimant's estimate of his actual needs, because of the difference in the capacity of claimants to make a just or accurate estimate; but a law which throws the door open to speculative claims and declares that a speculative absorber of streams can secure an adjudicated right thereto simply by asking for it is an iniquity. The Territorial water law was not framed for that purpose, but it nevertheless opened the door for the water grabber.

Thirty-one claimants to the water of Crazy Woman Creek filed these statements, some being for existing ditches and some for proposed ones. It does not appear from their character that there was any attempt to secure speculative rights, since some of the claimants asked for more water than they needed and others for less. The truth seems to be that the intention of all these irrigators was to comply with the requirements of the law in good faith, but that the volumes claimed had to them no definite meaning, and their statement was entirely unintelligible. There was probably not a farmer along the stream who knew whether he used 1 cubic foot of water or 10. Hence these claims exhibit chiefly the irrigators' lack of knowledge and the necessity, in an adjudication, of some impartial and adequate investigation of the facts on which any reasonable or just right should rest. It was the belief of the holders of these water rights, however, that the filing of this ex parte statement constituted in itself an absolute title to water, and that the further action in adjudicating these rights was only to confirm their title to the volume claimed. The outcome was in accordance with this belief. A comparison of the statements of claims filed in the county clerk's office with the volumes allotted in the decree shows that in nearly every instance they are the same. While a number of those who filed statements of claim do not appear in the decree, and a number of appropriators appear in the decree who did not file claims, there are enough appearing in both to show that the adjudication was almost if not entirely a proceeding to give the makers of these claims all they asked. The following table shows the agreement between the claims filed in 1886 and the appropriations established by the decree of 1889:

Comparison of appropriations under decree of 1889 with original claims filed.

Priority number.	Name of ditch.	Name of appropriator.	Volume allotted in decree, in cubic feet per second.	Volume claimed in statement.
2	John R. Smith	John R. and Agnes Smith	67.03	*a* 80
3	North Fork	Wyoming Land and Cattle Co.	30	55
4	Harris No. 1	W. Harris	12.2	12.2
6	Holt	Wyoming Land and Cattle Co.	8.20	8.20
8	Moreton	Moreton Frewen	22.32	22.32
9	Cook	Orrin Cook et al	33.55	33.55
10	North Fork of Crazy Woman.	J. A. Dowlin	28	28
17	Dick	Powder River Cattle Co	12.32	12.32
16	Kennedy	R. Kennedy et al	32	32
19	Blue Gap	Little North Fork Ditch Co.	26.5	26.5
19½	Teddy Miller	F. G. S. Hesse	84.45	84.45
20	Fort Collins	Fort Collins Land and Improvement Co.	210	210
21	Devoe No. 1	H. W. and C. M. Devoe	60.10	43
22	Devoe No. 2	do	78.11	78.11
25	Thompson & Matthews.	F. S. and G. F. Thompson	12	12
26	Mitchell & Long	B. Long and M. F. Mitchell	25	25
27	Harris No. 2	W. Harris	10	10

a 80 originally; 67.03 later.

If this creek carried enough water for all users, giving each appropriator all he claimed, the decree might not have been open to objection. Unfortunately the stream does not. It may for a few days or few hours each year carry 900 second-feet, although this is to be doubted; but during the latter half of the irrigation season the mean discharge is not one-tenth of this amount, and at times the channel below these ditches is a bed of dry sand. Not one-half the acreage designated in these appropriations has ever been irrigated, nor is there water enough in the stream to irrigate one-half of it without resorting to storage. With storage and with proper economy in the use of water all the land could probably be brought under cultivation, but to secure these there must be some method of restricting appropriators to their actual needs in the first place and some security for the control of stored water in the second. Extravagant appropriation is a prolific source of mischief in preventing both of these *desirable results. The* holder of a large appropriation claims the

right to control it whether he needs it or not. Parties hesitate to construct storage works to impound water which runs to waste when the title thereto is clouded, as it must be when more water than the stream carries at flood time is decreed to be appropriated.

These excessive rights have been an incessant source of trouble to the water commissioner. The holder of priority No. 2 can close all the ditches above him if his claim is recognized, and if it is not he has a grievance against the commissioner. Two instances have already occurred in which the assistance of the State engineer had to be asked to settle controversies over the right of these holders of a surplus to arbitrarily close the head gates of later appropriators. There is the further danger that these holders of extravagant rights will in time proceed to sell the surplus. If they have the right to do this, as the supreme court of the State has decided in a similar case, the time will come when the owners of the first three appropriations will practically control the stream, and the subsequent rights will not be worth considering. Meantime the water commissioner is forced to be judge, jury, and sheriff in the division of water among users. Under the limitation of the decree which gives to appropriators only the water needed, the commissioner refuses to close the head gates of subsequent appropriators, except in cases of actual need on the part of the holders of prior rights. But this policy is being continually called in question and could be carried out equally well without a decree.

DISTRICT 3.

This district, as shown in fig. 1, includes Clear Creek and its tributaries, and extends from the summit of Cloud Peak on the west to the junction of Clear Creek and Powder River on the east. It is one of the most important districts in the State and one of the most interesting to the student of water-right problems.

FLOW OF CLEAR CREEK.

Measurements of the amount of water flowing in Clear Creek were begun in 1889, their necessity having arisen from controversies over the water rights which the shortage of that year rendered acute. The location of the gaging station is about 4 miles above the city of Buffalo, as shown on fig. 2. The principal diversions above the gaging station are those in the mountains where two ditches carry water across a low divide into the head of French Creek. The measurements made at this point, therefore, do not represent the total flow of the stream, except at such times as the principal ditches are closed. The results of the measurements up to and including 1893 are not available; those for 1894 are shown diagrammatically on page xxxiii of the Second Biennial Report of the State Engineer of Wyoming and are given in the following table. During 1895 observations were

not made because it was impracticable to obtain the services of an observer. The results of the record for 1896 are published in Part

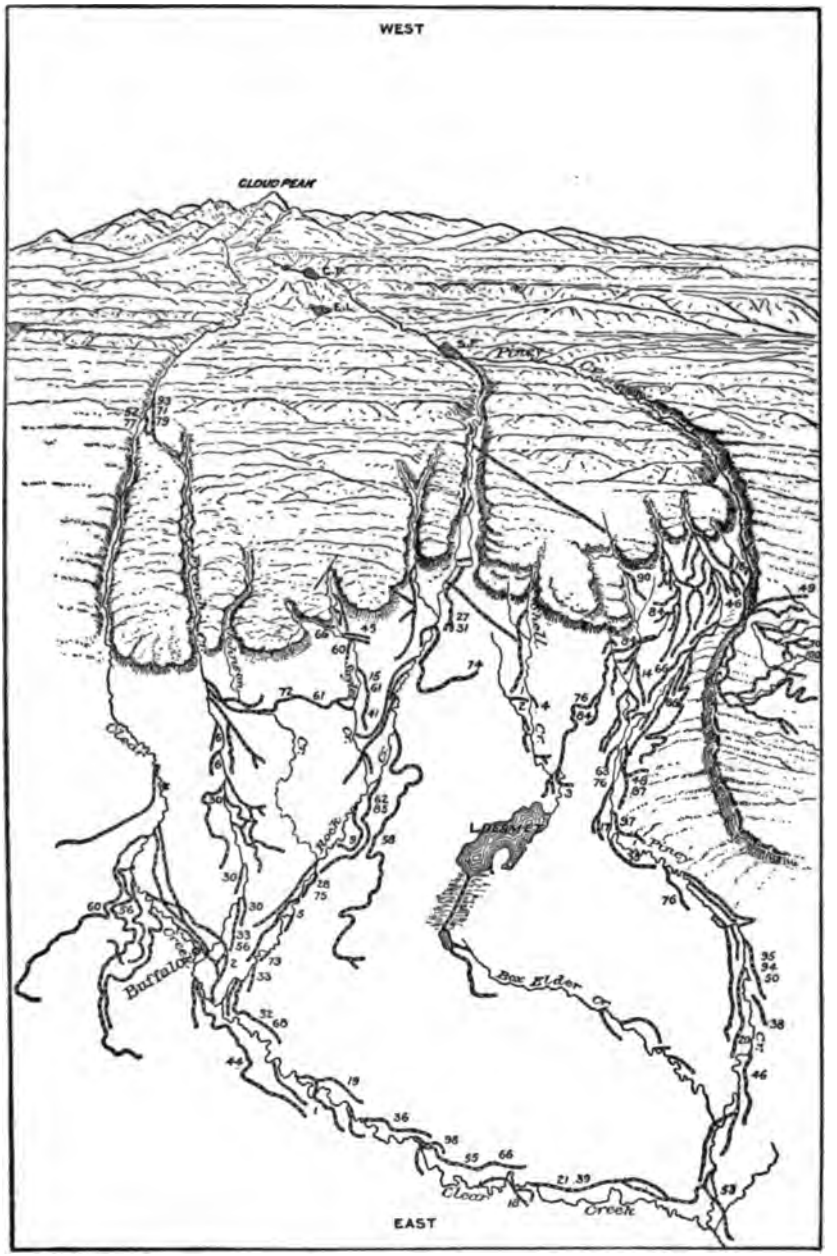


FIG. 2.—Irrigation systems on Clear Creek and tributaries.

IV of the Eighteenth Annual Report of the United States Geological *Survey, on page 140,* and those for 1897 in Part IV of the Nineteenth

Annual Report, of the Geological Survey, on page 298. The discharge for the three years is also given below and is shown diagrammatically in fig. 3 (page 31).

Estimated monthly discharge of Clear Creek at Buffalo, Wyoming.

[Drainage area, 118 square miles.]

Month.	Discharge.			Total.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth.	Per square mile.
1894.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Acre-feet.*	*Inches.*	*Sec.-feet.*
June	853	168	283	16,841	2.68	2.40
July	242	62	148	9,100	1.44	1.25
August	80	46	54	3,320	.53	.46
September	80	46	55	3,273	.53	.47
1896.						
May	406	64	139	8,547	1.36	1.18
June	504	82	251	14,936	2.37	2.13
July	323	64	115	7,071	1.12	.97
August	134	40	66	4,058	.64	.56
September	56	40	46	2,737	.44	.39
October 8–19	38	29	32	756	.12	.27
1897.						
May	632	209	345	21,214	3.37	2.92
June	657	189	280	16,662	2.64	2.37
July	246	55	112	6,887	1.09	.95
August	82	39	54	3,320	.53	.46
September	.....	.....	*a* 30	1,785	.28	.25
October	.....	.....	*a* 30	1,845	.29	.25
November	.....	.....	*a* 30	1,785	.28	.25
December	.....	.....	*a* 26	1,599	.25	.22

a Approximate.

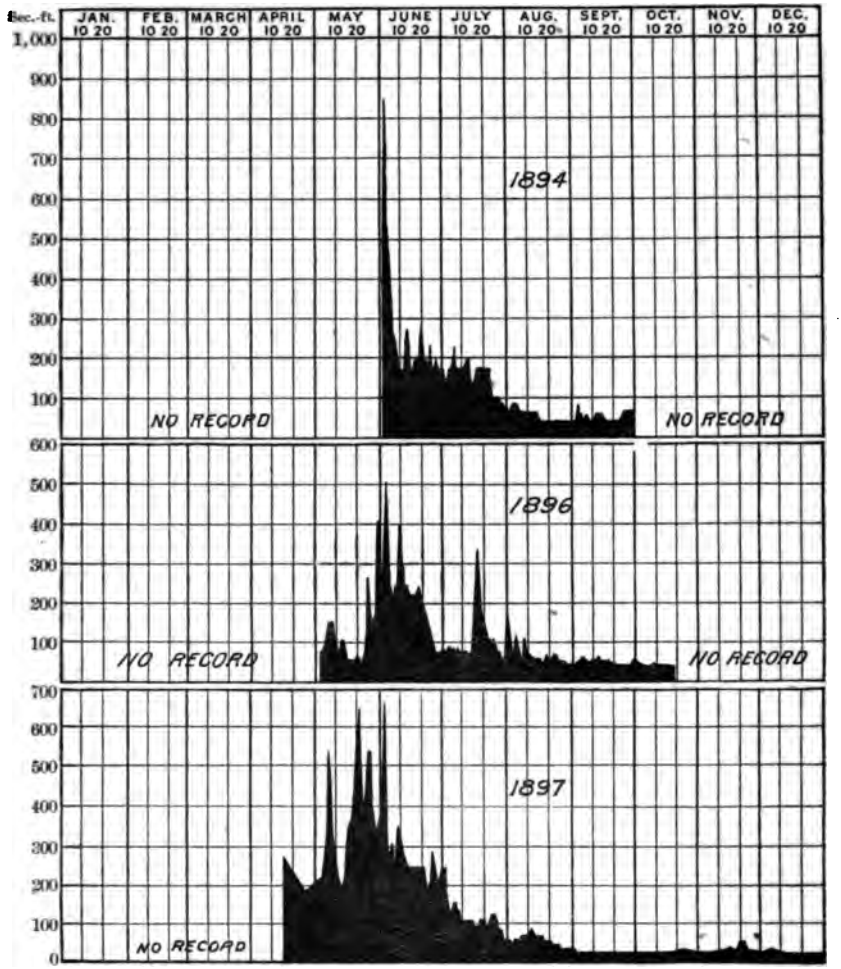


FIG. 3.—Discharge of Clear Creek near Buffalo, Wyoming, 1894, 1896, and 1897.

EARLY RIGHTS AND THEIR ADJUDICATION.

In May, 1878, Manlius Redmond began a ditch to water land along the main stream. It was, so far as the records show, the first attempt at irrigation in northern Wyoming, and the only attempt made that year. Ditches were built in 1879 along nearly every water course of the Bighorn Range, but in the previous year Mr. Redmond was alone. For more than ten years the building of ditches and the extension of the irrigated area made rapid progress, but 1889 was a season of drought, and the shortage on Clear Creek was so marked as to warn the users of water that the limit of profitable reclamation without the aid of reservoirs had about been reached. It also showed the need of an early determination of priorities. In the absence of this, position *was everything; the* ditch farthest upstream took its full supply

[illegible] of the rights or necessities of users below, while those [illegible] [illegible]

[illegible] the worst sufferers from this condition of affairs was the [illegible] Two large canals, shown in fig. 2, had been built to [illegible] the water of the North Fork of Clear Creek across a divide [illegible] French Creek. Their combined capacity was sufficient to drain the stream, and as no seepage returned, the loss to users below was [illegible] An adjudication of priorities was begun before the close of the year, but before the preliminary steps had been completed the Territory of Wyoming held a constitutional convention and adopted a provision placing the control of the public waters of the State in the hands of the board of control. The judge of the district court ruled that this action divested his court of jurisdiction, and further progress was stayed until after the admission of Wyoming to statehood and the passage of the State irrigation law.

Under the State law rights to water are determined by the board of control, and an adjudication was begun by measuring the ditches which take water from a stream and by making surveys of the land irrigated or which can be irrigated. The measurements and surveys of Clear Creek and its tributaries were begun in 1891; when completed they were placed in the hands of W. J. Clarke, the division superintendent, whose duty it was to take the testimony of appropriators. Mr. Clarke at once began the taking of testimony, but had not completed this when the board of control met, in February, 1892. On his return from this meeting the division superintendent, who was a stockman, became involved in the "invasion" of Johnson County, as it was called, and all the proofs submitted were lost. It was necessary therefore to begin proceedings anew and to take all the testimony over again. Before this work was completed the testimony formerly taken was recovered, and both sets of proofs were before the board of control at its final determination of these rights. The two sets of proofs disclosed many discrepancies and contradictions, and were the cause of several contests, which delayed the final determination of these rights until 1895.

DIVERSION ACROSS DIVIDES.

The map of Clear Creek and its tributaries (fig. 2), shows the location and priority numbers of the several ditches; it also shows that one of these tributaries, Piney Creek, drains a larger mountain area than the main stream and also has a greater discharge. In addition there are the smaller tributaries of French, Johnson, Sayles, Rock, Shell, J. A., Boxelder, Bear Gulch, and Little Piney creeks. Clear Creek and Piney Creek each reach to the summit of the range and drain the entire eastern slope of Cloud Peak, its highest elevation. The other streams are cut off from the perpetual snows and only drain the outlying foothills; hence, while they have an abun-

PINEY CREEK BELOW OUTLET OF CLOUD PEAK LAKE.

regardless of the rights or necessities of users below, while those farthest down got nothing.

Among the worst sufferers from this condition of affairs was the city of Buffalo. Two large canals, shown in fig. 2, had been built to carry the water of the North Fork of Clear Creek across a divide into French Creek. Their combined capacity was sufficient to drain the stream, and as no seepage returned, the loss to users below was absolute. An adjudication of priorities was begun before the close of the year, but before the preliminary steps had been completed the Territory of Wyoming held a constitutional convention and adopted a provision placing the control of the public waters of the State in the hands of the board of control. The judge of the district court ruled that this action divested his court of jurisdiction, and further progress was stayed until after the admission of Wyoming to statehood and the passage of the State irrigation law.

Under the State law rights to water are determined by the board of control, and an adjudication was begun by measuring the ditches which take water from a stream and by making surveys of the land irrigated or which can be irrigated. The measurements and surveys of Clear Creek and its tributaries were begun in 1891; when completed they were placed in the hands of W. J. Clarke, the division superintendent, whose duty it was to take the testimony of appropriators. Mr. Clarke at once began the taking of testimony, but had not completed this when the board of control met, in February, 1892. On his return from this meeting the division superintendent, who was a stockman, became involved in the "invasion" of Johnson County, as it was called, and all the proofs submitted were lost. It was necessary, therefore, to begin proceedings anew and to take all the testimony over again. Before this work was completed the testimony formerly taken was recovered, and both sets of proofs were before the board of control at its final determination of these rights. The two sets of proofs disclosed many discrepancies and contradictions, and were the cause of several contests, which delayed the final determination of these rights until 1895.

DIVERSION ACROSS DIVIDES.

The map of Clear Creek and its tributaries (fig. 2), shows the location and priority numbers of the several ditches; it also shows that one of these tributaries, Piney Creek, drains a larger mountain area than the main stream and also has a greater discharge. In addition there are the smaller tributaries of French, Johnson, Sayles, Rock, Shell, J. A., Boxelder, Bear Gulch, and Little Piney creeks. Clear Creek and Piney Creek each reach to the summit of the range and drain the entire eastern slope of Cloud Peak, its highest elevation. The other streams are cut off from the perpetual snows and *only drain* the outlying foothills; hence, while they have an abun-

PINEY CREEK BELOW OUTLET OF CLOUD PEAK LAKE.

dant supply for early irrigation, the late supply is wholly inadequate and, unless reenforced, will only serve for the reclamation of a restricted area. These smaller tributaries are, however, bordered by lands equal in area and of equal value with those along the valleys of the larger ones; hence some method of reenforcing their water supply became an early and important problem. The topography of this region offers unusual opportunities for doing this.

The head waters of French Creek are almost directly below Clear Creek, as its torrential flood comes from the rocky, snow-clad slopes of Cloud Peak. About all that is needed to turn Clear Creek out of its channel is a dam high enough to cause the water to overflow its bank, as the slope is toward French Creek from near its northern margin. One canal built to turn water into French Creek is only half a mile long, while the other, in a less desirable location, is only a mile in length. The map of Clear Creek and its tributaries (fig. 2) shows the location of these two canals, together with the priority numbers 71, 79, and 93 of their appropriations.

The turning of the water from Clear Creek into French Creek and the use of the channel of French Creek as a part of two canal systems created some perplexing problems for the board to solve in the determination of the two classes of rights from the latter stream. Both before and after the construction of the canals to provide this supplemental supply, parties had built ditches and appropriated the original or natural flow of French Creek. Some of these appropriators are interested in the canals in order to obtain an augmented supply, so that in a few cases the same individuals claimed rights to water from two sources through the same ditch to be used to irrigate substantially the same land. Other parties who made no claim to the natural flow of French Creek, but who were part owners in the canals to furnish the augmented supply, had built ditches from French Creek for the purpose of taking out the water turned into it.

There are, therefore, three classes of appropriators along French Creek. First, those who claim the natural flow of the stream; second, those who claim a right to the water of both Clear Creek and French Creek; and third, those who claim the right to use the channel of French Creek as a part of their ditch system, but draw their water supply entirely from Clear Creek. Among these different appropriators a difference of opinion exists as to the volume of water supplied by the natural flow of French Creek. At the time of the adjudication it was urged by the owners of the rights from Clear Creek that the claims of appropriators from French Creek were for more water than the creek carried, that the acreage originally irrigated had been extended by the unauthorized use of a part of the supply turned in, and that the board should restrict these claims. This was not done, because it was found impracticable to measure the flow of French Creek. To have done this would have involved shutting down the

ditches from Clear Creek and the ditches of appropriators above some of the tributaries of French Creek. A single measurement would not be sufficient. In fact, it would take a large number of measurements to reach any definite conclusion.

The demands of the area now under irrigation along the stream are so largely in excess of the natural flow of French Creek that to turn off the water from Clear Creek for a sufficient length of time to permit the measurement of the natural flow of French Creek would have caused the destruction of two-thirds of the crops along the stream. Those appropriators who have an interest in Clear Creek alone or in both supplies naturally objected to such action. The board was therefore compelled to rely on the engineer's surveys and on the proof submitted. Rights were restricted to the actual areas irrigated, but the source of the appropriation recognized was based on statements of irrigators. Those who claimed that their lands were irrigated from French Creek alone were given appropriations for their lands from that source, and those who claimed to have used only the water of Clear Creek were given a right from that stream. Whether or not this was in accordance with the facts will probably never be definitely known. It is one of the instances where haphazard development has reached a stage that renders any accurate settlement out of the question.

The same difficulty now confronts the water commissioner in performing his duties. The two ditches which divert Clear Creek into French Creek are so inaccessible as to make their frequent regulation impossible. Both are mountain ditches with head gates over 9,000 feet above sea level. Their channels are cut among the pines that cover this region and through the rocks from which they spring. To reach them requires a steep and toilsome journey of 15 miles from the mountain's base. They are at an elevation where frost is perpetual and snow almost so. The regulation of such ditches is, therefore, difficult and expensive, and any continuous measurement of the volume they carry is, for the present, out of the question, but the increasing value of these rights will in time render the accurate division of water indispensable. The water from Clear Creek mingles with the natural flow of French Creek until it reaches the first ditches built to divert it. There a part is taken out and carried over the divide between French Creek and Johnson Creek, and serves to reenforce the latter; the ditch is also extended across a second divide, where the remainder is turned into Sayles Creek to reenforce the flow of this stream and of Rock Creek below their junction.

The fluctuations in the flows of French Creek and Clear Creek do not agree, because the latter is fed from the summit of the range, while the former drains a larger area of the foothills region; it rises earlier in the season and also begins to subside earlier, but as the water turned in joins it near its source no measurements to deter-

A.

B.

RESERVOIR SITES ON PINEY CREEK.

A, At Elk Lake; *B*, At Mead Lake.

mine these variations can be had without an outlay greater than can now be afforded. The practical result is that during the early part of the season all users take from the natural flow of French Creek, while during the latter part nearly all are supplied from Clear Creek. The same conditions exist on Johnson and Sayles creeks, while on Rock Creek there is a third element of uncertainty. This, however, has arisen since the adjudication of rights, and will be described later.

PRAIRIE DOG CREEK.

Referring again to the drainage map of this district (fig. 2), it will be noticed that Piney Creek, which rises in the lakes on the northern

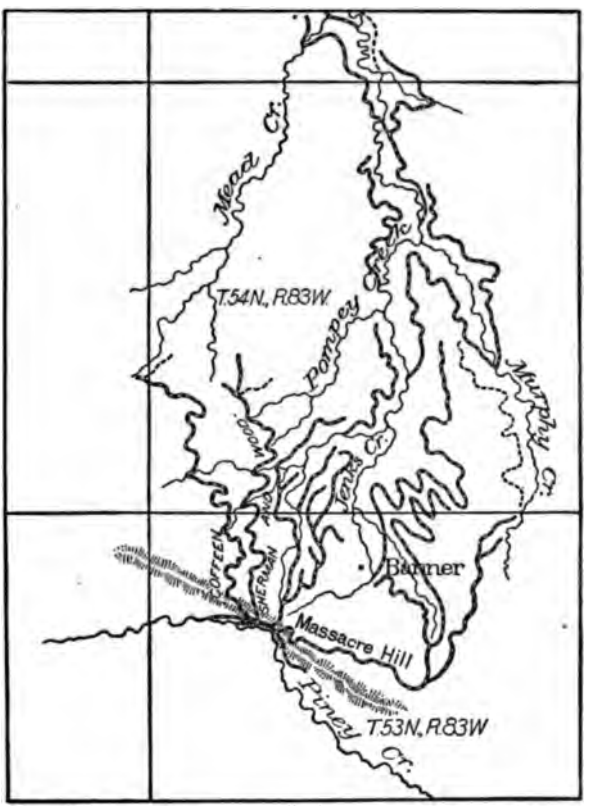


FIG. 4.—Ditches diverting water from Piney Creek across Massacre Hill divide to Prairie Dog Creek.

slope of Cloud Peak (Pl. II), not only drains a large extent of the summit of the range, but effectually cuts off a number of the smaller tributaries of Clear Creek therefrom. Piney Creek, like Clear Creek, receives nearly all its water from one direction. After the stream turns to the north the drainage on the east is into the smaller streams which rise in the foothills below. The most interesting peculiarity of the topography of this stream is, however, the fact that it

not leave the mountain in a valley, but on a ridge, the mouth of the canyon of Piney Creek being near the summit of the divide which separates the drainage of Tongue and Powder rivers. This divide is an elevated spur of the main range known as Massacre Hill. It leaves the mountain almost at a right angle and extends for a considerable distance eastward. At one point Piney Creek is within 50 feet of its summit. After that it tumbles down into the valley which borders its eastern slope.

Mead, Pompey, Jenks, and Murphy creeks, which form Prairie Dog Creek, a tributary of Powder River, rise on the northern slope of Massacre Hill almost directly opposite the point where Piney Creek so nearly approaches its summit, as shown in fig. 2. Originally this stream was of little importance. It lacked the snowfall necessary to a perennial water supply, and although a few springs are found along its channel, the volume available for irrigation was insignificant. It was an easy project, however, to carry water from Piney Creek over the divide, as shown in fig. 4 and in Pl. IV, and turn it into this stream. Three ditches have been built for this purpose. These ditches follow the prevailing practice of this region in making natural channels a part of the ditch system. The owners of one ditch live along Prairie Dog Creek for 15 miles. The water turned in at the head is taken out by small ditches which divert the creek near their farms. There are 84 appropriations through the ditch of the Prairie Dog Water Supply Company which are diverted from the second stream through a score of smaller ditches.

Before the supplemental water supply had been provided a number of settlers filed on land along Prairie Dog Creek and built ditches therefrom. Some of these settlers became part owners of the ditch to provide an additional water supply; others did not, so that the board again had to deal with three kinds of rights from the same channel—rights to the natural flow, rights to the water turned in, and rights to both combined.

There was a further complication in this case. Prairie Dog Creek is a part of the drainage system of Tongue River, and if drainage lines had to be regarded, it belonged to that district; but since practically the entire water supply came from Piney Creek, the board ruled that water supply rather than the slope of the country should govern its supervision, and this portion of the drainage of Tongue River was made a part of the Clear Creek district.

SUMMARY OF RIGHTS TO WATER.

The diagrammatic map of the irrigation system from Clear Creek (fig. 2) gives the location of the ditches, with the priority numbers of their appropriations. A full list of these is given in the Third *Biennial* Report of the State Engineer of Wyoming, 1895–96, pages *181 to 194.*

NORTH SLOPE OF MASSACRE HILL, ILLUSTRATING EROSION IN DITCHES FROM PINEY CREEK.

The amount of each of the appropriations was fixed by the acreage shown by the proof to have been irrigated, 1 cubic foot per second being allowed for each 70 acres. On Crazy Woman Creek it will be remembered the appropriations were based on the claims filed by appropriators, without regard to the actual use of the water. On French Creek appropriations were based on the areas reclaimed, without regard to the volume of water described in the original statement of claim. The relation of these claims to the actual use of water in irrigation is shown by the comparison of the volume claimed and the volume appropriated on French Creek, as given in the table below.

Volumes appropriated on French Creek as fixed by the order of the board of control and volumes claimed in appropriators' statements.

Priority number.	Name of ditch.	Name of appropriator.	Volume appropriated in decree, in cubic feet per second.	Volume claimed in original statement, in cubic feet per second.
1	Sneider No. 1	Mary L. Carpenter et al	1.64	8
2	Moeller No. 1	Alice S. Rapelyen	1.07	6
2	Moeller No. 2	do	.50	3
3	Moeller No. 3	do	4.28	13.97
4	John A. Fisher No. 1	John A. Fisher	.50	3.35
4	John A. Fisher No. 2	do	.43	23.15
4	Mayhew & Gorgen	Robert Foote	.17	12.40
5, 7	Dundee	do	1.57	8.2
6	Gorgen	Peter Gorgen	.28	13.97
	Total		10.44	88.04

It will be seen from the totals that the appropriations established by reason of beneficial use amounted to only about one-tenth of those claimed. The ratio on all of the streams of this district could not be determined, because many of the claims were for the "unappropriated waters of the stream," designating no specific volume; but if these claims to whole streams were excluded, the claims for a definite volume alone were largely in excess of the actual use, as will be seen by the following comparison, in which all of the claims for indefinite amounts have been thrown out.

lumes claimed and volumes actually used on streams in water district No. 2, in cubic feet per second.

Name of stream.	Volume used.	Volume claimed.
Total Clear Creek	232.78	853.6
Total Rock Creek	187.64	749.9
Total Sayles Creek	8.18	75.1
Total Johnson Creek	3.13	44.7
Total Piney Creek	334.88	943.8
Total Shell Creek	5.52	68.6
Grand total	772.13	2,735.7

ALLOWANCE FOR LOSS OF WATER IN TRANSMISSION.

Among the interesting questions raised in this adjudication was the extent of the loss of water from canals by seepage and evaporation and the authority of the board to provide for this by an increase in the appropriation.

In the original order of the board no such allowance was made, it being held that 1 cubic foot per second for each 70 acres is so much in excess of the requirements of crops, when the water is distributed with reasonable skill and economy, that it will provide for all necessary loss in transportation.

The Wyoming Land and Cattle Company was, however, dissatisfied with this decision, and on this and other grounds asked for and obtained a rehearing, at which measurements were presented showing the losses taking place between the head gate and the place of use.

The conditions on this ditch are somewhat unusual and need to be stated in order that the results may be properly interpreted. The head gate of the ditch is on what was formerly the Fort McKinney military reservation, near the mouth of the canyon and 3 miles above the town of Buffalo. At the time of these measurements, irrigation from the ditch or above the ditch was not permitted within the reservation limits. After crossing its boundary the ditch passes for several miles around and across a series of ridges on the slopes of which losses by seepage are considerable. In the ravines between these ridges and, in one place, along a slope which the ditch crosses there are beds of sand where the loss from percolation is so great that flumes have been built at these points. For 10 miles the following conditions favor an exceptional loss:

The land through which the ditch is built is not irrigated and local atmospheric conditions favor rapid evaporation.

It is the highest ditch and therefore gains nothing from the irrigation of lands above.

The arid land, the steep slopes, and the stretches of sand all add to the drain by percolation.

The measurements submitted were made in 1896 and 1897 by Fred Bond, a civil engineer of Buffalo, Wyoming, and were made in the following manner: A rectangular weir 4.75 feet in length was placed in the ditch immediately below the head gate, and a similar weir, with a length of 5.75 feet, was placed in the ditch near the lands to be irrigated, 10 miles from the first weir and 56 feet below it, that being the fall of the ditch in this distance. It was found by experiments that it took from eleven to thirteen hours for water to pass from one weir to the other. Hence in making these tests the measurements at the upper weir were taken in the morning and those at the lower weir in the evening, in order to measure as nearly as possible the same supply. The tables which follow give the results obtained, that for August 27 being evidently due to an error in measurement or to disturbance by storms.

Loss of water in transmission through ditch of Wyoming Land and Cattle Company.

Date.	Depth of water on upper weir, in feet.	Discharge, in cubic feet per second.	Depth of water on lower weir, in feet.	Discharge, in cubic feet per second.	Total leakage in flumes, in cubic feet per second.	Total loss, in cubic feet per second.	Loss, in per cent.
1896.							
July 10	0.83	11.448	0.56	7.865	(a)	3.583	31.3
July 11	.82	11.328	.55	7.642	(a)	3.686	32.6
July 13	.76	10.123	.55	7.642	(a)	2.481	24.5
August 25	.65	8.058	.50	6.876	(a)	1.182	14.7
August 27	.62	7.515	(?) .58	(?)9.270	(a)	.....	.....
August 28	.60	7.152	.46	5.877	(a)	1.275	17.9
August 29	.59	6.986	.45	5.671	(a)	1.315	18.9
August 30	.59	6.986	.44	5.547	(a)	1.439	20.6
August 31	.59	6.986	.45	5.671	(a)	1.315	18.9
September 1	.58	6.799	.44	5.547	(a)	1.252	18.5
1897.							
August 8	0.562	6.37	0.34	3.74	0.068	2.56	40.2
August 9	.567	6.69	.34	3.74	.09	2.86	42.8
August 10	.60	7.31	.36	4.06	.125	3.12	42.8
August 11	.68	8.59	.43	5.27	.08	3.24	37.7
August 12	.63	7.70	.38	4.42	.047	3.23	42.0
August 24	.467	5.99	.24	2.24	(a)	3.75	62.6

a Not measured.

It will be seen by examining these tables that the percentage of loss when a small volume of water was carried was much greater than when the depth was increased, seepage and evaporation being apparently about constant. If the board took the maximum loss of over 60 per cent as a basis for an order increasing the original right so as to give 1 second-foot for each 70 acres irrigated at the place of use, it would result in nearly double the original appropriation, while if the minimum loss of less than 15 per cent were made the basis of a new right, this action would require so slight a change as not to warrant a modification of the order.

The board refused to grant the additional allowance. The rehearing established a right to water for an additional acreage and increased the volume to be turned into the ditch. As the loss from seepage and evaporation seems about constant, and does not materially increase when the ditch is full, the larger the amount turned in the smaller the percentage of loss. Hence, with the larger right a minimum loss of less than 15 per cent may be expected.

Fort McKinney has since been abandoned and the reservation will soon be largely irrigated. It is believed that the percentage shown by the measurements will decrease in the future by the building of ditches above this one and the irrigation and saturation of the land below it.

The allowance of 1 second-foot to 70 acres being in excess of actual needs was regarded as sufficient to meet the average loss from the ditch at the present time and ample to meet the ultimate decrease.

The wide variation between percentages in the months when the use is heavy and in the later ones when the use is small, would make the establishment of a satisfactory or just appropriation to meet this loss difficult, if not impossible, while the probability of a progressive decrease in such loss would make it unwise to interfere. Even if the proportion allowed agreed with the conditions when made there is small likelihood of such agreement five years hence.

The board held, therefore, that the making of appropriations in excess of the statutory limit would not be approved; but that where satisfactory proof of unpreventable losses in transit made it apparent that the volume remaining, when the land was reached, would not produce crops, a temporary increase would be provided for by an administrative order to the water commissioner.

OWNERSHIP IN DITCH NOT NECESSARY TO THE ACQUIREMENT OF A WATER RIGHT.

One of the parties submitting proof of the beneficial use of water from Piney Creek was John Leitner, who showed to the satisfaction of the board that he had irrigated and reclaimed 15 acres of land through the Upper Phil Kearney ditch. The board by order estab-

lished the right to water for this land through said ditch. In his proof, Leitner stated that he was a part owner of the ditch through which the appropriation was made, and this claim was not contested in the proceedings before the board. Subsequent litigation arose over the ownership of the ditch, and the court decided that Leitner had no interest in it, but that Thomas J. Foster was the sole owner. Following this decision Foster requested the board of control to amend its records by making him the owner of the water right for 15 acres of land acquired by Leitner through the use of the water in its irrigation. Foster's request was based on the contention that his ownership of the ditch made him the owner of all the water it diverted or of all the rights to water acquired through it.

As it is the belief of the board that rights to water for irrigation, in the State of Wyoming, do not inhere either in the man who filed the claim or in the ditch which carried the water, but in the land reclaimed, the request for a transfer of the right from both the place where acquired and from the man who acquired it did not seem well founded; but recognizing the importance of the question involved, the board, before rendering a decision, asked the advice of the Attorney-General. The letter of inquiry and the reply were as follows:

CHEYENNE, WYO., *March 20, 1896.*

DEAR SIR: The State board of control is in receipt of a petition for review of the order determining the priorities of water from Piney Creek and tributaries, and requesting that the board of control cancel a certain certificate of appropriation of water to John Leitner, guardian, and find that Thomas J. Foster is the sole owner of the Upper Phil Kearney ditch.

The board of control has given careful consideration to this petition, and to the order of the court determining the ownership of the Upper Phil Kearney ditch, and has examined the evidence submitted before said board in the adjudication referred to. We find that the testimony submitted showed that Leitner had irrigated the land for which an appropriation was issued through the Upper Phil Kearney ditch and was entitled to an appropriation therefor, and after a reexamination of the evidence the board is of the opinion that the decree establishing this appropriation was correct.

In awarding an appropriation for the particular land for which an appropriation was claimed by said Leitner, the board did not attempt to determine that he had any ownership whatever in the ditch of Mr. Foster. It simply decided that the evidence submitted showed that the water had been transported through this ditch. The board has always held that its authority is restricted to the examination and determination of the facts showing the application of the water of the State to beneficial use and the rights thereby created, and that it could not determine the property rights in ditches through which the water was conducted. That question has been before the board repeatedly and the decisions have been uniform. In the same way the board is now of the opinion that it is not within its jurisdiction to say that Mr. Leitner is or is not a part owner in the Foster ditch. The decree of the court is regarded as determining that fact, but its validity or effect will not be in any way enhanced, in our opinion, by an assumption on the part of the board that it has authority to in any way determine such facts.

We have, by order, instructed the division superintendent to notify the water commissioner of the effect of the order of the court, and that the amount of the

appropriation through the Upper Phil Kearney ditch has been reduced by such order by the amount necessary for the irrigation of 15 acres, but we do not feel authorized to cancel the certificate of Mr. Leitner. We believe if he can secure satisfactory arrangements for the transportation of the water appropriated through the Upper Phil Kearney ditch by giving suitable compensation to Mr. Foster for the service rendered, or if he can make arrangements for the transportation of the water appropriated for this land through some other ditch, he has a perfect right under our laws to do so, and that the appropriation for the land is not destroyed by the fact that he is not part owner in a particular ditch.

Recognizing the importance of the questions raised, and desiring to avail ourselves of your assistance in their proper solution, we shall be pleased if you can advise us as to whether or not, in your judgment, the board has any authority whatever to determine or pass upon property rights in ditches, and whether or not the decree of the court in this case makes it mandatory upon us to cancel the certificate of appropriation of said Leitner.

Respectfully submitted.

ELWOOD MEAD,
President State Board of Control.

Hon. B. F. FOWLER,
Attorney-General, Cheyenne, Wyo.

MARCH 23, 1896.

DEAR SIR: Replying to your communication of March 20, with reference to the application of Thomas J. Foster for a review of your decision granting a certificate of appropriation to John Leitner, guardian, in which application for review Mr. Foster claims to be the sole owner of the Upper Phil Kearney ditch, and presents a decree of the district court of Johnson County, in an injunction suit brought by Mr. Foster against Mr. Leitner, the decree establishing the ownership of Mr. Foster:

I desire to state that while the State board of control was correct in respecting the decree to the extent of instructing your water commissioner that the amount of the appropriation for the Upper Phil Kearney ditch should be reduced by the amount of water granted therein to Mr. Leitner for the irrigation of fifteen acres, still at the same time you would not be justified in canceling the certificate issued to Mr. Leitner.

While there are allegations in the petition for review to the effect that Mr. Leitner made fraudulent representations to your board with regard to his having an interest in the ditch in question, still, in the absence of an affirmative showing of such fraud, there would certainly be no authority on the part of your board to take this for granted and cancel the certificate in question. The decree of the district court of Johnson County does not indicate this, nor do I understand that affidavits or other proof have been filed at this time for this purpose. The petition which is filed is not in the nature of a petition for rehearing, and I am of the opinion that this question can not be properly reached by your board in any other manner, and unless with the application for a rehearing proof should be presented to your office showing that at the time Mr. Leitner used the Phil Kearney ditch he did so as a trespasser and without the consent of Mr. Foster, then your certificate should stand.

As to whether or not Mr. Leitner is able to use the water in the future is certainly not a question to be considered, as under the statute he is given two years before he can be charged with abandonment, and if during that time he should build a new ditch or arrange with Mr. Foster or some other person to conduct the water from the stream in question upon his land, he would be entitled to appropriate enough to irrigate fifteen acres.

As to the claim that there is a cloud upon the title to Mr. Foster's ditch by your granting the certificate, this is certainly a mistake, and as Mr. Foster would not get the benefit of the water for the fifteen acres which is appropriated by Mr. Leitner, I can certainly see no advantage to Mr. Foster in securing the cancellation of the certificate in question.

Respectfully submitted.

BENJAMIN F. FOWLER,
Attorney-General.

Hon. ELWOOD MEAD,
State Engineer, Cheyenne, Wyo.

The reply confirmed the board's views and the request was refused, Mr. Leitner being given two years in which either to build a new ditch or to arrange with Mr. Foster for the transportation of the water to which he was entitled through the Upper Phil Kearney ditch.

DISTRICT 4.

This district, shown in figs. 1 and 5, includes Big Goose Creek and its tributaries. Little Goose Creek is by far the most important tributary, the acreage irrigated along its valley being greater than along the valley of the main stream. Among the smaller tributaries, Soldier, Rapid, and Beaver creeks are the principal branches of Big Goose Creek, while Jackson, Hanna, Tepe, Davis, Trabing, Kemp, Sackett, and Kruse creeks are all branches of Little Goose Creek, which aid in augmenting its flow and from which lands are irrigated. Here, as in the Clear Creek district, the principal stream drains the summit of the range and cuts off the late water supply of the smaller tributaries, and here also irrigators along these smaller tributaries have taken advantage of the topography of the country to relieve their shortage by carrying water across the divides. This transfer of water from one stream to another has been carried to such an extent in this district that some users get water from half a dozen sources. Irrigators near the lower end of Little Goose Creek have the natural flow of the stream, water from Piney Creek, and from three ditches from Big Goose Creek, making in all five separate sources of supply. These transfers from one watershed to another and this extended use of streams as parts of ditch systems have undoubtedly greatly extended the acreage of land reclaimed, but it has at the same time enormously increased the difficulty of determining and protecting rights. Determining the rights to one stream is hard enough, because of the uncertainty and conflicts of early records, but when these uncertainties embrace four or five sources the problem is infinitely more complex. Take as an illustration the rights to water through the Last Chance ditch. Its head gate is in Little Goose Creek. Whenever there is a surplus in this stream irrigators take from the nearest source. Whenever the supply is inadequate a ditch owned by the same persons diverting water from Big Goose Creek is opened and water is used from that stream. These *irrigators* have therefore established a right to water

from both streams. The question which confronted the board was, How to frame a decree which would give to these irrigators the right to take water from both streams and at the same time limit it to a single appropriation. If it was absolutely certain that water rights would remain attached to the land where acquired and the decree could have been so worded as simply to give this land whatever water was

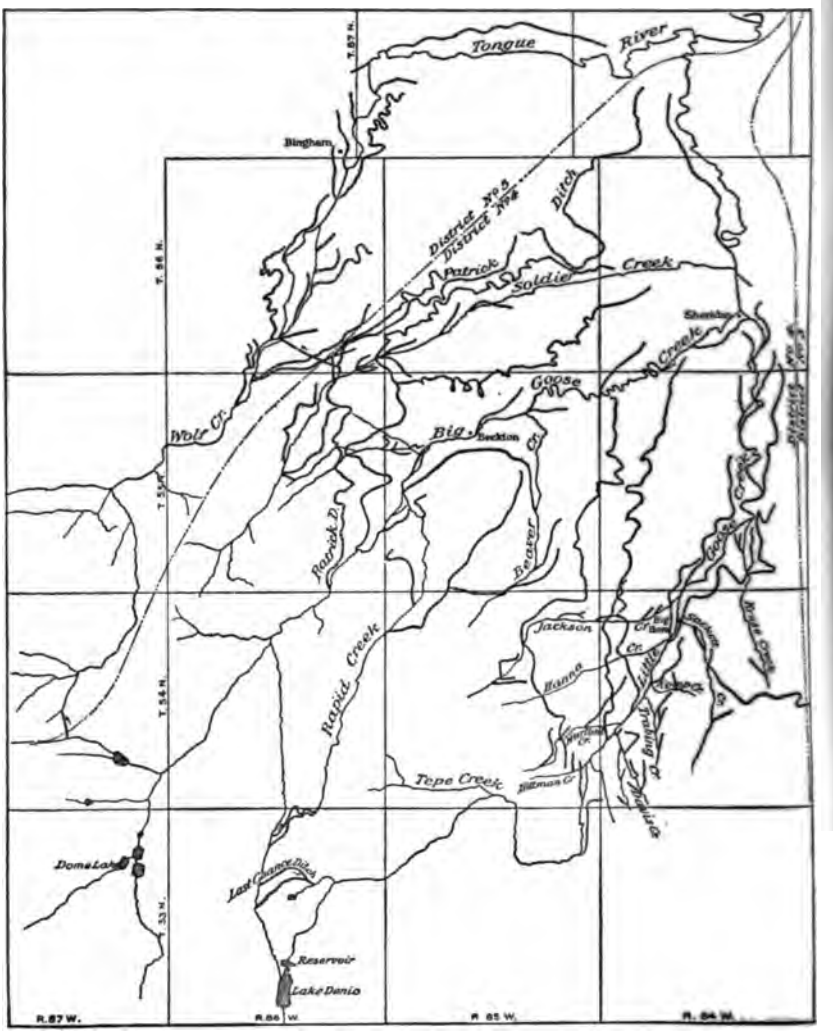


FIG. 5.—Location of ditches on tributaries of Tongue River, Division II, district 4.

needed, there would have been little trouble in preparing the certificates, but the statutes of Wyoming prescribe the form of certificates of appropriation and require the board issuing them to give the volume appropriated in cubic feet per second continuous flow. It was necessary to give the same number of second-feet from each stream, and to

TIMBER FLUME IN CANYON OF TONGUE RIVER

this continuous, since there is no authority in the law for limit-
e appropriation to certain months or to certain portions of the
ion season. The board endeavored, in framing its order, to
it any claim to a double appropriation being set up in the
. But its success will largely depend upon the decision of the
ne court as to whether rights acquired under the Territorial law
sold and transferred to other lands. If this right should be
l, there is no question that the right to dispose of the appropria-
rom one of these streams will become a matter of litigation.
use of two ditches to divert a single appropriation and the use
atural stream as a part of the canal system made it difficult to
orities. Many of the ditches from Little Goose Creek were con-
ed prior to those which turn water from Big Goose Creek and
lifferent priority numbers. Hence the same tract of land has a
d appropriation—the two parts coming from different sources
aving different priorities.
boundaries of district No. 4 are based on drainage lines, and
o the beginning of irrigation there was no connection between
eams of this district and those on either side above the mouth
Goose Creek. There are three places at which water is trans-
from one district to another. It is turned from the North
of Big Goose Creek into the head of Wolf Creek in water district
and by the Patrick ditch from Big Goose Creek across Soldier
, and thence into Wolf Creek at a point about 12 miles farther
and on the south water is turned into Little Goose Creek by
the canals from Piney Creek, so that drainage lines are being
obliterated. When the Granger canal is completed there will
ntinuous connection by ditches from Powder River on the south
Montana boundary, a distance of 100 miles, uniting four districts.
mary of the appropriations of water in this district is given in
st Biennial Report of the State Engineer of Wyoming, 1891–92,
xxv to xxx, and continued in the Third Biennial Report, 1895–96,
177 and 180.

DISTRICT 5.

listrict No. 5, shown in figs. 1 and 6, the cost of ditches, rather
he adequacy of the water supply, governed the location of the
first reclaimed. Tongue River, the main stream, has still a
volume unappropriated, which each year runs to waste, but on
ber of its tributaries there are more lands under ditch than these
is will fertilize. This condition of affairs is due to the fact that
building from Tongue River is expensive, as shown in Pl. V.
the stream leaves the mountains the grade of its channel is very
and the valley narrow. On both sides are high bluffs with very
rrigable land along their slopes, while to reach the table-lands
requires long, crooked ditches with many dikes or flumes, where
s or tributary streams have to be crossed.

The shortage of water on the tributaries led to early quarrels over its division. Litigation over water rights on Wolf (east of area shown on fig. 6), Amsden, Smith, and Hewes (or Columbus) creeks began when Wyoming was a Territory, but no conclusion had been reached when the State law went into effect. The testimony which had been taken in this litigation was transferred to the board of control, and these streams were among the first to be dealt with by that body. Since the priorities on these tributaries were earlier than the appro-

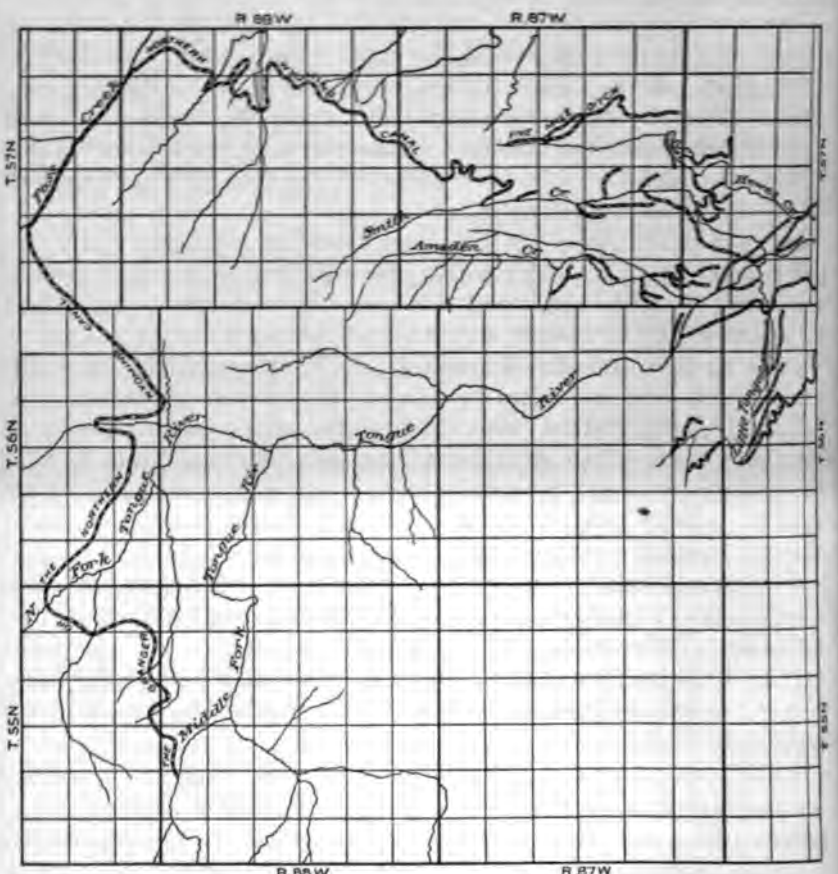


FIG. 6.—Location of ditches on North Fork of Tongue River, Division II, district 5.

priations on the main stream they were independent of any claims on the latter. Hence, in the final determination of these rights, the board of control dealt with each as a separate and distinct drainage basin. The announcement of the policy of the board of control, in limiting water rights to the volume actually used, was not in accord with the views of some of these claimants, who held that their statements of claims described the amounts of their appropriations, and that to reduce these was an encroachment on vested rights.

n appeal from the board's order was taken by the second appro-
,tor from Amsden Creek. This appropriator was allowed 0.86 of
ıbic foot per second for the irrigation of 60 acres of land, the loca-
of the land being described. The appeal was based on the claim
; water for 150 acres had been appropriated, and made it neces-
r for the court to rule on the question as to what, under the Wyo-
g law, constituted an appropriation. The following extract from
proof of appropriation gives the testimony on which the board
ted the right to water for 60 acres:

ıestion. State the number of acres watered each subsequent year, and give
egal subdivisions on which the water is used.
ıswer. Claimant increased the amount of land irrigated each year from 1882
ıe present time, and now has about 60 acres under cultivation and irrigation.
ıestion. State the acreage said ditch is capable of watering, and show on a
or plat, on a scale of 2 inches to the mile, the location of said ditch and the
it is capable of watering.
ıswer. One hundred and fifty acres.
EMARKS.—Claimant appropriated water sufficient for 150 acres of said land, and
claims water for said land. He does not use said water, has not heretofore,
if farming becomes more profitable he may desire to cultivate all of his land,
his appropriation was made to cover all of said land.

will be seen from this testimony that only 60 acres had actually
ı watered, but that the claimant stated it was his intention at
e future time, when irrigation became more remunerative, to
er more land. The map made by the State engineer's office showed
; about 100 acres of land belonging to the claimant could be
ered, but that about 50 acres of land belonging to other parties
ld be watered from this ditch, so that the claim for a possible irri-
on of 150 acres from the ditch was correct. The maps and proofs
ther claimants showed that about 435 acres of land had been
aimed, and the actual necessities of this land had more than
austed the water carried by the stream, so that the resulting short-
had led to litigation over these rights prior to the admission of
oming to statehood. This lack of an adequate water supply was
wn by the gagings made by the State engineer's office, the volume
July being 5.25 cubic feet per second, while the aggregate of the
ropriations made by the board of control on the restricted basis
he necessities of the land reclaimed was 6.21 cubic feet per second.
he claim of an appropriation for 150 acres, instead of 60, had been
gnized by the board, the water for the additional 90 acres would
e been taken away from lands already reclaimed and from present
rs, to be held as a speculative property by this claimant until it
ame valuable enough for him to use, or until, trading on the
essities of those now using it, he could compel them to purchase
om him.
he evils of basing appropriations of water on such considerations
ld not *have ended with this* individual. If his belief that he would

at some future time require water enough for 150 acres of land constituted an appropriation, the belief of any appropriator on a stream that he might at some future time use more water would also have entitled him to the full amount of his probable necessities, and appropriations would be fixed not by physical conditions, but by the state of men's minds when filing claims. The first appropriation on this stream was for water sufficient to reclaim 100 acres, but the owners of that ditch had, prior to the adjudication, enlarged it, and had actually reclaimed 110 acres additional, for which they received at the hands of the board of control an appropriation having a later priority. If, however, the contention of the appealing appropriator had been sustained, the appropriator holding the first priority could have maintained that he had intended to appropriate the whole stream and have taken it, while the other five appropriators, including the party appealing from the board's decision, would have been compelled to look to him rather than to the mountains for their water supply.

It seems strange that such an indefinite and unreasonable basis for water rights should ever have had any defenders or received serious consideration; but until the court sustained the board's ruling public opinion in the State was very evenly divided as to the legality of the action. The reason for this lies in the fact that the evils of these extravagant grants are not at once manifest. The term "cubic foot per second" is, to the majority of irrigators, wholly meaningless, and this fact made them unable to appreciate either the effect of an allowance of their claims or the adequacy of the appropriation which the board of control had approved. When a claim had been made for 20, 50, or 100 second-feet of water, and the claimant found that it had been cut down to 1, 2, or 3 second-feet, it was but natural that he should feel aggrieved and be inclined to look with distrust on the assurance that this volume was entirely sufficient to meet all his requirements. The result of basing appropriations on the volumes claimed from Amsden, Smith, and Hewes creeks is not, however, a matter of speculation.

On the last-named stream twenty-eight parties submitted proofs of appropriations, and the surveys of the engineer's office showed that each of them had used water. The aggregate volume of the appropriations approved by the board was 34.8 second-feet, which was less than half the 76 second-feet claimed by the first five. Subsequent experience has shown that the volume fixed by the board's order is more than the stream will furnish, and some of the later appropriators are now building a ditch from Tongue River; but if the stream does not carry 34 second-feet it certainly would not have supplied 76 second-feet, and the allotment of this volume to the first five appropriators would have left nothing for the remaining twenty-three. It is manifest, also, that they have never used more than the stream *carries*, and of course it is impossible to acquire an appropriation

without ever having diverted or used the water. The injustice of making these claims the foundation of adjudicated rights is strikingly shown in the case of this stream. The claim of the first appropriator was for 21 second-feet, and of the second for 10 second-feet, so that if these two had been given all they asked they would practically have owned the stream.

Smith Creek did not supply enough water to meet the actual needs of users when the priorities were determined. The order of the board limited the rights of all the appropriators to 13.97 cubic feet per second. This was probably in excess of the volume actually used, but it was a great reduction from what was claimed, since that of the first appropriator alone was for 150 second-feet, or more than ten times the volume which the stream carries. The claimant of 150 second-feet received 6 feet, and was indignant because he believed that he had been unjustly deprived of the difference.

LITTLE TONGUE RIVER.

The rights to water from Little Tongue River, the principal tributary of Tongue River, have not as yet been determined. At the time of the adjudication there had not been a sufficiently thorough investigation of the physical conditions to enable the board to reach a satisfactory conclusion as to the merits of some of the questions which must be decided. One of these is a physical problem of great interest. It is a well-known fact that all of the streams draining the eastern slope of the Bighorn Mountains lose more or less water through underground channels. One of the branches of Powder River sinks, to reappear below with diminished volume. The losses on the South Fork of Clear Creek amount to a considerable percentage of its entire flow during the latter part of the irrigation season, and a ditch built to turn water therefrom into Crazy Woman Creek has been a failure, because the water diverted disappeared. But of all these streams the loss in Little Tongue River is the greatest.

During an examination made five years ago a number of funnel-shaped openings were discovered, through which large volumes of water were disappearing, with no evidence of their reappearance. In one case the volume was measured and was found to be in excess of 3 cubic feet per second. In the places examined the water sinks where streams cut across the upturned edges of a limestone stratum of great thickness. Whether the total volume lost from Little Tongue River is greater than that from the main stream is as yet unknown, but the percentage is greater and the loss more apparent. Those dependent on the stream have made many efforts to ascertain the location of these underground channels and close them up, and much has been accomplished in this direction. One of the parties engaged in this work claims that the water thus saved should not be subject to the rights of appropriators of the normal discharge, but

should be wholly independent thereof. In his claim for a separate right and for its adjudication apart from the rights on the main stream he sets forth by affidavit the facts on which the claim is based. The affidavit is here given:

John R. Gatewood, being first duly sworn, deposes and says that he is a citizen of the county and State aforesaid; that he is the person described in the annexed "Application for a permit to divert and appropriate the waters of the State of Wyoming," and that the water which he desires to appropriate from the South Fork of the Tongue River, one of the natural streams of the State, is water which has not as yet been appropriated by any person for "beneficial uses;" that in the said South Fork of Tongue River, back in the mountains, at places described in said "application," there are natural holes on the bottom of the stream through which about 20 per cent of the natural flow of said water, at low water, escapes into the earth, and that, as far as affiant has been able to ascertain, does not again run into said stream or any other natural stream, but is wholly lost; that said affiant believes that he can save said water for beneficial uses; that he is the owner of possessory rights to agricultural lands in said county, and has not water to reclaim the same, and that by being allowed to save this water he verily believed, and does believe, that he can secure said water for said beneficial uses.

A list of the adjudicated rights in water district No. 5 is given in the First Biennial Report of the State Engineer of Wyoming, 1891-92, pages xxxiii to xxxvi and pages xxiii and xxiv. These tables are continued in the Third Biennial Report, 1895-96, page 176.

APPROPRIATIONS IN DIVISION III.

On the west side of the range but little has been done by the State irrigation authorities. Rights have been determined in the sixth district, but not in the fourth and seventh (see fig. 1), there being so large a surplus as to obviate any controversy among users.

DISTRICT 6.

The following order and tabulation gives the rights established in the sixth district and the form in which the entry of the board's action is made in the records of the board of control:

ORDER ESTABLISHING PRIORITIES AND AMOUNTS OF APPROPRIATIONS OF WATER FROM NO WOOD RIVER AND ITS TRIBUTARIES.

In the matter of the determination of the rights of appropriators of water from No Wood River and tributaries, in water division No. 3, Bighorn County, Wyoming.

In this matter the board of control having heretofore, at its regular meeting held on the 21st day of March, 1896, duly made and entered an order for the adjudication and determination of the rights to the use of the waters of No Wood River and its tributaries, in water division No. 3, and directing the State engineer to proceed as soon as possible and survey and measure all the ditches diverting water therefrom;

And it appearing that the State engineer, in pursuance of said order, after giving due and legal notice according to law to all appropriators from said stream and its tributaries, did survey and measure all the said ditches, and prepared and filed complete maps and a report in the office of the said board:

Thereafter N. H. Brown, superintendent of said water division No. 3, after due

and legal notice to appropriators of water fixing the time and place of receiving proof of appropriation from said stream and its tributaries, did appear at the time and place fixed, and did take and receive the sworn proofs and testimony in writing of all persons appearing and claiming rights to water in said stream, and thereafter exposed said proof to public inspection on a day fixed according to law, and said proofs having been examined and no contests having been instituted were thereafter duly filed, with his report and findings, in the office of said board;

And now this matter coming regularly on for hearing and determination by the board of control, upon the records in the State engineer's office and the proofs of appropriation duly taken and submitted by said superintendent, and upon the maps and report of surveys and measurements of streams and ditches duly made by the State engineer and his assistants, and the board having now examined the records in the State engineer's office, the proofs of the beneficial use of water, filed in the office of said board, and having read the report and tabulation and heard and considered the recommendations of said N. H. Brown, superintendent of said division No. 3, that certificates of appropriation be issued, and the board, being now fully advised, finds as follows:

FINDINGS.

The board finds that the appropriations of water determined and established by this order were made from the following-named streams:

No Wood River proper and Bear Creek, Boxelder, Upper or Little Canyon Creek, Crooked Creek, Otter Creek, Spring Creek, Tensleep Creek, Broken Back Creek, Alkali Springs Creek, Paint Rock Creek, all tributaries of No Wood proper.

Cherry Creek, tributary of Boxelder.

North Fork Spring Creek, tributary Spring Creek.

Lower Canyon Creek, tributary Tensleep.

North Fork Broken Back, tributary Broken Back.

Buffalo Flat Creek, tributary Alkali.

Medicine Lodge Creek, tributary Paint Rock.

Coon Holler or Coon Creek, tributary Spring.

That in order to adjudicate and establish the rights of the various appropriators from these several streams it is necessary, in addition to the determination of the order of priority and general priority number of each of the rights to water from No Wood and its tributaries, taken as a whole, for the board to determine and establish the priorities of right growing out of appropriations from each of said streams as between the appropriators therefrom themselves.

The board finds that the rights of appropriators from each of the various tributaries are subject to the rights of prior appropriators on the main stream below, or on the stream into which said tributary flows, and to these rights only.

Subject to the foregoing limitations and conditions, the board finds the name of the appropriators, the priority numbers of said appropriations, the maximum amounts thereof, the uses for which the water was appropriated, and the legal subdivisions of land on which the water appropriated is to be used and to which it is attached are as set forth in the following table.

ORDER.

It is therefore ordered by the board of control that [illegible] appropriators from No Wood and its tributaries [illegible] water of the stream from which said appropriation was made [illegible] the beneficial purpose for which acquired and for [illegible] the appropriators, the priority number of each of said appropriations [illegible] mum amounts thereof, the names of the streams or ditches [illegible] which the appropriation is made, the date of said appropriations [illegible] use to which the water is appropriated, and when for irrigation [illegible]

of the land to which the water appropriated is attached, shall be the same as stated in the following table under its appropriate heading, which said table is hereby made a part of this order.

It is further ordered that each of the appropriations for irrigation shall be sufficient for the economical irrigation of the land described and no other, not to exceed 1 cubic foot of water per second of time for each 70 acres irrigated.

NO WOOD RIVER PROPER.

Priority number.[a]	Name of ditch.	Name of appropriator.	Post-office address.	Date of appropriation.	Use to which water is applied.	Amount appropriated, in cubic feet, per second.	Number of acres irrigated.
5 1	S. V	John D. Gill	Redbank, Wyo.	May 1, 1885	Irrigation	0.79	55
11 2	Cornell No. 1	Enoch Cornell	do	Apr. 2, 1886	do	.90	30
12 3	Mead No. 1	John Mead	do	May 1, 1886	do	.15	10
38 4	Spratt	Thos. Spratt	Bonanza, Wyo.	May 13, 1889	do	3.21	225
41 5	Green Spot	Chas. Mills	Redbank, Wyo.	1889	do	.15	10
41 5	Cornell No. 2	Enoch Cornell	do	1889	do	.12	8
42 6	Suez	Taylor Bros	Bonanza, Wyo.	Mar. 15, 1890	do	1.86	130
43 7	Harmony	O. C. Morgan	do	Mar. 20, 1890	do	2.28	160
43 7	do	Henry Jordan	do	do	Domestic and irrigation.	.88	60
43 7	do	Louisa M. Smith	do	do	do	.21	15
43 7	do	A. G. Smith	do	do	do	.53	37
43 7	do	Jas. A. Quiner	do	do	do	.74	50
46 8	Umslopogaas No 2.	A. L. Coleman	Redbank, Wyo.	Nov. 1, 1890	do	.64	45
49 9	Western	L. J. Diehl	Bonanza, Wyo.	Aug. 2, 1892	do	2.78	195
49 9	do	Nich. Carstensen	Hyattville, Wyo.	do	do	1.14	80
49 9	do	Geo. A. Bell	do	do	do	4.08	286
53 10	Ilg	E. M. Ilg	Bonanza, Wyo.	Oct. 16, 1893	do	.71	50
53 10	do	Otto Ilg and Arthur A. Ilg.	do	do	do	.68	48
60 17	Avent	Ira M. Avent	do	July 5, 1895	do	1.71	120
62 18	Cornell No. 2, enlargement.	John Mead	Redbank, Wyo.	Sept. 16, 1895	do	.21	15
63 19	Harmony Mill enlargement.	Henry Jordan	Jordan, Wyo.	Jan. 4, 1896	Mechanical	1.15	.....

BOXELDER CREEK, TRIBUTARY OF NO WOOD RIVER.

1 1	Two Bar	Estate of Harvey Booth.	Redbank, Wyo.	Apr., 1884	Irrigation	0.57	40
16 2	Helms No. 1	Henry Helms	do	Apr., 1886	do	.40	28
21 3	Wells No. 1	Chas. Wells	do	May, 1887	do	.46	32
24 4	Wells No. 2	do	do	Sept. 1, 1887	do	.63	44
25 5	Helms No. 2	Henry Helms	do	Sept., 1887	do	.31	22

UPPER OR LITTLE CANYON CREEK, TRIBUTARY OF NO WOOD RIVER.

13 1	Red Bank	H. A. Richards	Redbank, Wyo.	June, 1886	Irrigation and stock.	4.57	320
40 2	Umslopogaas	A. L. Coleman	do	May, 1889	Irrigation	.50	35
45 3	Rosebud	Harvey Smith	do	Sept., 1890	do	.10	7
55 4	Hillside	C. S. Walu	do	Feb. 16, 1894	do	.93	65
55 4	do	Alex. Arledge	do	do	do	.50	35

CROOKED CREEK, TRIBUTARY OF NO WOOD RIVER.

8 1	F. D. Helmer, or Brunner No. 1.	Frank D. Helmer	Redbank, Wyo.	May 15, 1885	Irrigation	1	70
8 1	Ainsworth	F. S. Ainsworth	do	do	Irrigation and mechanical.	1.14	80

[a] *The first* number is the general priority number in reference to the rights to water from *No Wood River* and its tributaries taken as a whole; the second is the priority number in *reference to* the rights from each stream.

OTTER CREEK, TRIBUTARY OF BIG WOOD RIVER.

Priority number.		Name of ditch.	Name of appropriator.	Post-office address.	Date of appropriation.		To what water is applied.	Amount of water in cubic feet per second.	Number of acres irrigated.
4	1	Grout	C. E. Shaw	Bellevue, Wyo.	Apr.	1865	Irrigation	1.0	[illegible]
7	2	Dyson	do	do	Mar.	8, 1865	do	[illegible]	[illegible]
28	3	Daly	do	do		1867	do	[illegible]	[illegible]
31	4	Higbie	C. J. Higbie	do	Mar.	1868	do	1.0	[illegible]
58	5	Sawyer	George Sawyer	do	Feb.	1884	do	[illegible]	[illegible]

SPRING CREEK, TRIBUTARY OF BIG WOOD RIVER.

5	1	Emge & Robinson No. 2	Wm. Emmons	Bellevue, Wyo.	May	1865	Irrigation	1.0	[illegible]
5	1	Walz Bros.	R. A. Walz	do	do		do	1.0	[illegible]
16	2	Emge & Robinson (Hollywood).	Wm. Emmons	Tensleep, Wyo.		1866	do	[illegible]	[illegible]
17	3	Standish & Henderson.	Elmer Thatcher	do	Mar.	31, 1867	do	1.0	[illegible]
35	4	Tunnel	A. F. and L. Walz	do	Sept.	1868	do	1.0	[illegible]
52	5	Tunnel enlargement, No. 2.	R. A. Walz	do	Sept.	1883	do	[illegible]	[illegible]

TENSLEEP CREEK, TRIBUTARY OF BIG WOOD RIVER.

2	1	Winn	Milo Burke	Tensleep, Wyo.	Sept.	1864	Irrigation	1.0	[illegible]
7	2	Columbian	Milo Burke and Bessie C. Burke	do	May	31, 1865	do	1.0	[illegible]
10	3	Bay State No. 1	Bay State Cattle Co.	do		1865	do	1.0	[illegible]
22	4	Burke	Milo Burke	do	Aug.	1867	do	[illegible]	[illegible]
30	5	Perfection	Fred G. S. Hesse	Buffalo, Wyo.	Apr.	31, 1869	do	[illegible]	[illegible]
44	6	Bay State No. 2	Bay State Cattle Co.	Tensleep, Wyo.	May	1880	do	[illegible]	[illegible]
48	7	Wyman	S. A. McLaughlin	do	July	1882	do	1.0	[illegible]

BROKEN BACK CREEK, TRIBUTARY OF BIG WOOD RIVER.

7	1	Carothers No. 2	Ed. Carothers	Hyattville, Wyo.	May	1866	Irrigation	[illegible]	[illegible]
10	2	Cranky Jack	John D. Hopkins	Tensleep, Wyo.		1866	do	[illegible]	[illegible]
18	3	Hardscrabble	W. R. Williams	do	Mar.	1867	do	[illegible]	[illegible]
28	4	Carothers North Fork.	Ed. Carothers	Hyattville, Wyo.		1867	do	[illegible]	[illegible]
28	4	Carothers No. 1	do	do		1867	do	[illegible]	[illegible]
28	4	Carothers No. 3	do	do		1867	do	[illegible]	[illegible]
29	5	North Fork	John D. Hopkins	Tensleep, Wyo.	Apr.	1868	do	[illegible]	[illegible]
41	6	Dry Fork	H. E. Miller	do		1869	do	[illegible]	[illegible]
47	7	Carothers No. 5	Ed. Carothers	Hyattville, Wyo.		1870	do	[illegible]	[illegible]
47	7	Carothers No. 6	do	do		1870	do	[illegible]	[illegible]
47	7	Carothers North Fork No. 1.	do	do		1870	do	[illegible]	[illegible]
57	8	Williams	W. R. Williams	Tensleep, Wyo.	July	1884	do	[illegible]	[illegible]

PAINT ROCK CREEK, TRIBUTARY OF BIG WOOD RIVER.

6	1	Big Bear	David Matthews	Hyattville, Wyo.	May	1865	Irrigation	[illegible]	[illegible]
6	1	do	John Lemmon	do	do		do	1.0	[illegible]
14	2	Meyers	H. W. Meyers	do	Sept.	1868	do	[illegible]	[illegible]
14	2	do	John Wertz	do	do		do	[illegible]	[illegible]
14	2	do	Lee Nash	Bonanza, Wyo.	do		do	[illegible]	[illegible]
19	3	Luman & Allen	John Lemmon	Hyattville, Wyo.	Apr.	1867	do	[illegible]	[illegible]
19	3	do	Jas. D. Allen	do	do		do	[illegible]	[illegible]
23	4	Bernstein	Clara F. Collins	Basin, Wyo.	[illegible]	1867	do	[illegible]	[illegible]
27	5	Elk	Abraham Gowan	Hyattville, Wyo.	Feb.	1867	do	[illegible]	

PAINT ROCK CREEK, TRIBUTARY OF NO WOOD RIVER—Continued.

Priority number.		Name of ditch.	Name of appropriator.	Post-office address.	Date of appropriation.	Use to which water is applied.	Amount appropriated, in cubic feet, per second.	Number of acres irrigated.
27	5	Elk	John Luman	Hyattville, Wyo.	Fall, 1887	Irrigation	2.64	185
27	5	do	Jas. D. Allen	do	do	do	1.43	100
28	6	Bernstein No. 1	Estate of Fred Bernstein, D. B. Stanton, agent.	Basin, Wyo	1887	do	5	350
33	7	Military	T. F. Nelson	Hyattville, Wyo.	May, 1888	do	.54	38
37	8	Go Ahead	Willis J. Booth	do	May 1, 1889	do	1.79	125
37	8	do	H. D. Hendricks	do	do	do	1.57	110
37	8	do	T. F. Nelson	do	do	do	.43	30
50	10	Shuck & Wilson	Zelotes Wilson	do	Mar. 20, 1893	do	.79	55
50	10	do	Asa Shuck	do	do	do	1.71	120
54	11	Rinehart	Dan H. Rinehart	do	Jan. 23, 1894	do	.18	12
56	12	Hyattville	S. W. Hyatt	do	July 2, 1894	do	1	70
56	12	do	D. W. Hunsinger	do	do	do	.86	60
59	16	South Side	Lee Nansell	Bonanza, Wyo.	July 5, 1895	do	.43	29.3
59	16	Weintz	John Weintz	Hyattville, Wyo.	do	do	1	70
59	16	do	H. W. Meyers	do	do	do	.51	36
59	16	do	Nora O. Meyers	do	do	do	.50	35

CHERRY CREEK, TRIBUTARY OF BOXELDER CREEK.

61	1	Helms No. 3	Henry Helms	Redbank, Wyo.	July 11, 1895	Irrigation	.26	18

LOWER OR BIG CANYON CREEK, TRIBUTARY OF TENSLEEP CREEK.

3	1	Hunsinger No. 1	Jacob Hunsinger	Tensleep, Wyo.	Oct. 15, 1894	Irrigation	2.28	160
15	2	Hunsinger No. 2	do	do	Oct. 1, 1886	do	2.64	185
51	3	Canyon Creek Ditch.	Frank M. Simmons (C. E. Shaw, agent.)	do	Sept. 4, 1893	do	3.64	255

BUFFALO FLAT CREEK, TRIBUTARY OF ALKALI CREEK.

36	1	Buffalo Flat	John J. Smith	Hyattville, Wyo.	1888	Irrigation	.93	65

MEDICINE LODGE CREEK, TRIBUTARY OF PAINT ROCK CREEK.

9	1	George Bayne	D. V. Bayne	Hyattville, Wyo.	May 10, 1885	Irrigation	3.19	223
9	1	do	Estate of George Cassell, F. P. Carr, administrator.	do	do	do	2.53	177
10	2	Wickwire	Byron Wickwire	do	1885	do	.36	25
20	3	Allen & Nelson	Theodore F. Nelson.	do	Apr. 10, 1887	do	1.10	77
20	3	do	Jas. D. Allen	do	do	do	.87	61
26	4	Highland	Lorenzo D. Walu	do	Nov., 1887	do	.49	34
26	4	Walu No. 2	do	do	do	do	.57	40
32	5	Hyatt No. 2	Saml. W. Hyatt	do	May 10, 1888	do	1.21	85
34	6	Anthony	Byron Wickwire	do	Sept. 20, 1888	do	.08	5
34	6	do	David Matthews	do	do	do	3.50	245

COON HOLLER OR COON CREEK, TRIBUTARY OF SPRING CREEK.

39	1	Dutch	John J. Smith	Hyattville, Wyo.	May 17, 1889	Irrigation	.26	18

It is further ordered that certificates of appropriation be issued to the said appropriators in accordance with the provisions of the foregoing findings and order.

In witness whereof, the president of the State board of control has hereunto set his hand this 10th day of March, A. D. 1898.

RESERVOIRS.

By drawing on the larger streams when the smaller ones were exhausted it has been possible to utilize a large percentage of the flow from the east side of the range without resorting to storage, but the time is approaching when storage will be necessary if the reclaimed area is to be extended. Storage has been somewhat delayed, because there is no law at present defining and protecting rights to stored water. This is one of the legislative questions to be dealt with in the near future. Much can be done whenever it becomes necessary. The sites for mountain reservoirs are both numerous and valuable. From the summit of Cloud Peak over 100 mountain lakes can be counted, and a large percentage of these will in time be utilized.

Actual construction has begun on only one of these reservoir sites—Dome Lake. The location of this lake is shown in fig. 5, and a detailed sketch of the lake and surroundings is given in fig. 7. The elevation of the lake is 8,720 feet, as determined by the topographers of the United States Geological Survey during the season of 1897. The area of the water surface is 320 acres, the average depth 20 feet, and the total flow of water 6,400 acre-feet. The estimated cost of the reservoir is $50,000. The improvements have been made with the double object of storing water for irrigation and of making this point an attractive summer resort.

The importance of the lakes on the head waters of Piney Creek is clearly shown in the able report of Capt. Hiram M. Chittenden, giving the results of his preliminary examinations of reservoir sites in Wyoming and Colorado.[1] The relative location of these lakes is shown on the Cloud Peak topographic atlas sheet of the United States Geological Survey, the area being mapped during the field season of 1898. From a report by Mr. François E. Matthes, the following facts concerning South Piney Creek have been obtained:

The South Fork of South Piney Creek flows from the snow banks and glaciers east of Cloud Peak, and has therefore an abundant water supply throughout the year. Its valley, above the point where the ditch to Rock Creek begins, is the bed of an ancient glacier, and is thickly covered with morainal deposits of irregular disposition, giving rise to a remarkable number of lakes of all sizes. Among these are Cloud Peak Lake and Mead Lake, shown in fig. 8, both of which

[1] Preliminary examination of reservoir sites in Wyoming and Colorado, by Capt. Hiram M. Chittenden, with monograph on Reservoirs and their effects on the floods of the Mississippi system, by James A. Seddon: *House Doc. No. 141, Fifty-fifth Congress, second session.* Washington, 1898. *110 pp., 10 folding maps.*

have at various times been proposed for reservoir sites. Both are deep and are surrounded by steep mountain sides; the altitude of Cloud Peak Lake is 9,703 feet and of Mead Lake 9,950 feet, approximately. (See Pl. III, *B*.) Unfortunately but one of the two main canyons on the east side of Cloud Peak drains into these lakes. The other (lesser) canyon joins the main canyon half a mile below Cloud Peak Lake.

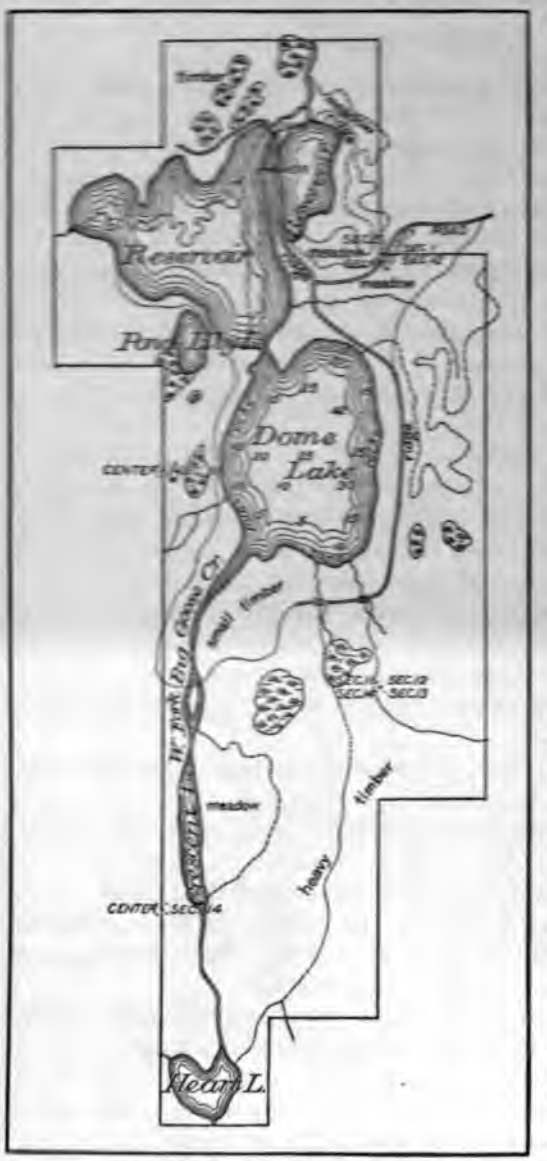


FIG. 7.—Dome Lake reservoir.

Another large body of water, Elk Lake, draining into the South Fork of South Piney Creek, lies on a broad shelf and is very shallow. The lake is at an elevation of 9,850 feet. Its catchment area is very small, almost entirely above timber line, and contains nothing larger than insignificant rills. In spring it probably receives a large amount of water from the melting snowdrifts on the peaks. The outlet of the lake is wide and flat and of a swampy character. The stream at that point is quite small, but increases farther on. It apparently sinks into the ground on leaving the lake, a phenomenon quite common in a country covered with glacial débris. By building a low dam the area of the lake will be considerably increased, but the water supply will be limited practically to that obtained in spring from snow banks which entirely disappear each summer.

The North Fork of South Piney Creek flows through a valley similar

ıat of the South Fork. The number of lakes, as shown ın fig. 8, 'en larger than on that branch, but few are available for water-ıge purposes. Among these are the Kearney Lakes, altitude 6 feet, in reality one lake, which, however, is fast being converted

FIG. 8.—Lakes at head of Piney and Goose creeks.

two separate lakes, as shown on Pl. VII, by the sediment of two ll brooks entering near the middle, opposite each other. The ı lakes thus formed are deep and can easily be dammed at the out- They receive all [redacted] om the glaciers in the canyons north

of Cloud Peak and that from the perpetual snow banks on the peaks of the main range. It is expected that surveys for a reservoir dam at these lakes will be commenced early next season.

The northern forks only of Clear Creek were mapped during 1898, but a general survey of the entire drainage area revealed the fact that each of the half dozen canyons which form its head waters contains a number of lakes, some of them almost as large as the Kearney Lakes. The entire basin is thickly covered with morainal ridges; it is extremely rough and practically inaccessible.

Sites along Tongue River will serve to increase greatly the acreage that stream will reclaim, while on the western slope the improvement of the lakes on Medicine Lodge and Tensleep creeks will serve not only to double the reclaimed area but to call attention to a region of unsurpassed scenic attractions.

The maps filed in the State engineer's office serve to show the location of the projects on which work was begun and the nature of the improvements proposed.

Names, superficial area, and contents of the reservoirs in the Bighorn Mountains for which permits have been issued by the State engineer.

Name.	Superficial area in acres.	Contents in acre-feet.
Dana reservoir	4	25
Kearney reservoir	146.2	3,801
Dome Lake reservoir	211.2	3,168
Willow reservoir	57.8	650
Barr reservoir	48	450
Treichler & Parsons reservoir	5.1	61.2
Ella reservoir	45	1,000
Denio reservoir	193.75	1,743.75
Cross & Cruse Co. reservoir	21	.....
Elk Lake reservoir	197.55	2,570
Cloud Peak reservoir	170	2,500
South Piney reservoir	95.4	1,050

ADMINISTRATIVE QUESTIONS.

The commissioners in charge of the districts described in this bulletin have to deal with unusual conditions. The division of a stream among users is a difficult problem under the most favorable circumstances, but the commissioners of these districts are confronted with complications seldom encountered in equal measure elsewhere. These are:

1. The interchange of water between streams in the same district *and its transfer* across district boundaries.

OUTLET OF BEARGRASS LAKE RESERVOIR ON NORTH FORK

2. The frequent use of streams as part of a ditch or canal and making not one stream but three or four streams part of a single system.

3. The attaching to one piece of land of rights from more than one stream through one or more ditches.

4. The construction of canals to carry water across divides in remote and inaccessible locations in the mountains, thereby rendering the regulation of head gates expensive and difficult and the measurement of stream flow practically impossible.

Even when the streams are gaged and the commissioner knows the supply to be divided he is often unable to perform his work to his own satisfaction or that of irrigators. The wide and rapid fluctuation of streams is to blame for this. From the records of the gagings of Clear Creek, made by the State engineer and the Geological Survey, it is seen that the discharge ranges from three or four times the total volume appropriated to about one-third this volume, and that at times these two extremes are reached within a few days of each other.

The gaging station is below the mountain ditches, and its record does not show what they divert, but it does show how rapid must be the commissioner's movements to protect the rights to the 232 second-feet appropriated. On May 23, 1896, as shown in fig. 3, less than 100 second-feet was passing over the weir; on the 24th there was over 200. On June 3 a flow of 504 second-feet permitted all users to waste water, if they so desired. Three days later the discharge was less than the amount appropriated. On the 19th the flow was 209 second-feet; on the 29th it was 82. The distance from the highest to the lowest head gate on this stream is 50 miles, and an examination of the map (fig. 2) will show the haphazard location of the appropriations and the rapid changes necessary to protect these rights when the demand begins to exceed the supply. The commissioner would have enough to do in times of scarcity to regulate this stream only, but he has in addition to watch the fluctuations of half a dozen others and keep track both of their natural discharge and of the water turned in from other streams.

While doing this he can only estimate the volume to be distributed. From the highest ditch on Clear Creek to the foot of the mountain is a day's journey. It takes half that time to go from the highest land irrigated on Rock Creek to the head gate of the Piney feeder in the mountains along that stream. From the last ditch on Prairie Dog to the first on the stream which supplies it the distance is 40 miles in one direction, and it is an equal distance to the last ditch along the valley of that stream in almost the opposite direction. Without telephone or telegraph connection, the commissioner can deal only with what he sees. Under the circumstances the success achieved has been little less than marvelous.

The water turned from Big Goose Creek into Little Goose Creek enters that stream 9 miles from and 2,000 feet above the land first

irrigated. The head of the ditches can only be reached after a day's journey, and frequent inspections can not be made. Because irrigators can not know the facts, they are all the more likely to resent any interference with their supply. Appropriators of the natural flow are sometimes disposed to believe that those dependent on some other source get more than their share, and vice versa. Users of water along Piney Creek, Big Goose Creek, and other large streams do not look with favor on the diversions into other drainage basins where the subsequent regulation is so expensive and difficult. It is, therefore, a high tribute to the honesty and tact of the commissioners and superintendent to say that they have thus far performed their duties in such a way as almost entirely to avert litigation. To enable them to continue this, however, their labors should be supplemented by comprehensive measurements of the discharge of these several streams. The turning of water across divides and the use of a stream as part of a ditch is no doubt economical, but unless the methods so largely employed here are supplemented by numerous stream gagings, it is hard to see how commissioners are to distribute successfully a supply coming from three or four sources.

Lack of gagings add to the perplexities of the State engineer. It is a part of his duty to refuse permits for additional ditches when the water supply is exhausted. So long as he has no knowledge of the volume being diverted, nor where it goes, he has to depend largely on conjecture. The extensive construction of reservoirs along the channels of these streams which is certain to result in the near future, and the use of their channels to transport the stored water to the place of use, will aggravate these complications. That they can all be removed is not to be doubted, but to accomplish this it will be necessary to have a full understanding of all the conditions, and foremost among these is the amount of water there is to be stored, divided, and used.

INDEX.

Page.

Administrative difficulties, discussion of . 58-60
Amsden Creek, appropriations from 46-48
Barr reservoir site, description of 58
Big Canyon Creek, appropriations from . 54
Big Goose Creek, appropriations from ... 43-45
map showing lakes at head of 57
Bighorn Mountains, plate showing view of 11
Billy Creek ditch, irrigation from 19, 20, 23
Blue Gap ditch, irrigation from 20, 23, 27
Board of control, work of 14-16
Bond, Fred, stream measurements by 39
Boxelder Creek, appropriations from 52
Broken Back Creek, appropriations from . 53
Buffalo, stream measurements near 30-31
Buffalo Flat Creek, appropriations from . 54
Burton ditch, irrigation from 20, 23
Canton ditch, irrigation from 20, 23
Chittenden, H. M., cited 55
Clear Creek, measurements of 28-30
map showing irrigation systems on .. 29
diagram showing discharge of 31
appropriations from 31-32, 38
diversion of water from 33-35
Cloud Peak, plate showing view of 11
Cloud Peak Lake reservoir site, description of 55-56, 58
Cook ditch, irrigation from 20, 23, 27
Coon Creek, appropriations from 54
Crazy Woman Creek, appropriations from 18-28
Crooked Creek, appropriations from 52
Cross & Cruse Co. reservoir site, description of 58
Dana reservoir site, description of 58
Denio reservoir site, description of 58
Devoe ditches, irrigation from 20, 23, 27
Dick ditch, irrigation from 20, 23, 27
Dome Lake reservoir, plate showing 54
description of 55, 58
map showing 56
Dundee ditch, irrigation from 37
Duty of water, amount of 24-25, 38
Elk Lake reservoir site, plate showing .. 34
description of 56, 58
Ella reservoir site, description of 58
Fisher ditches, irrigation from 37
Fort Collins ditch, irrigation from 20, 23, 27
French Creek, appropriations from ... 33-35, 37
Gorgen ditch, irrigation from 37
Harris ditches, irrigation from 20, 23, 27
Hewes Creek, appropriations from 46, 48

Page.

Holt ditch, irrigation from 20, 23, 27
Johnson Creek, appropriations from 38
Kearney Lake reservoir site, description of 57, 58
plate showing view of 58
Kelly Creek ditches, irrigation from 20, 23
Kennedy ditch, irrigation from 20, 23, 27
Kingsbury ditches, irrigation from 20, 23
Land reclaimed, acreage of 12-13
Last Chance ditch, appropriations from .. 43-45
Little Canyon Creek, appropriations from 54
Little Goose Creek, appropriations from . 43-45
map showing lakes at head of 57
Little Tongue River, map showing ditches from 46
appropriations from 49-50
Loss of water in transmission, allowance for 38-40
Lower Canyon Creek, appropriations from 54
Massacre Hill, plate showing view of 36
Matthes, F. E., cited 55
Mayhew & Gorgen ditch, irrigation from . 37
Mead Lake reservoir site, plate showing . 34
description of 55-56
Medicine Lodge Creek, appropriations from 54
Miller ditch, irrigation from 20, 23, 27
Mitchell & Long ditch, irrigation from . 20, 23, 27
Moeller ditches, irrigation from 37
Moreton ditches, irrigation from 20, 23, 27
Muddy Creek ditch, irrigation from 20, 23
Newell, F. H., letter of transmittal by ... 9
No Wood River, appropriations from 50-52
North Fork ditches, irrigation from 19, 20, 23, 27
North Fork of Crazy Woman ditch, irrigation from 20, 23, 27
Otter Creek, appropriations from 53
Paint Rock Creek, appropriations from .. 53-54
Piney Creek, plate showing view of 32
plate showing reservoir sites on 34
diversion of water from 35-36
map of ditches diverting water from . 35
appropriations from 38
map showing lakes at head of 57
Prairie Dog Creek, appropriations from . 35-36
map of ditches diverting water to 35
Px ditch, irrigation from 20, 23
Red Bark ditch, irrigation from 20, 23

61

Page.
Redmond, Manlius, irrigation by..... 12, 31
Reservoir sites, description of..... 55-58
Rock Creek, appropriations from 38
Sayles Creek, appropriations from..... 38
Shell Creek, appropriations from 38
Smith Creek, appropriations from..... 46, 48, 49
Smith ditch, irrigation from..... 19, 20, 23, 27
Smith *v.* Devoe decree, extract from... 18-19, 23
criticism of 21-28
Sneider ditch, irrigation from..... 37
South Piney reservoir site, description of..... 56-57, 58
Spring Creek, appropriations from 53
Storage, necessity of..... 27, 55
Tensleep Creek, appropriations from..... 53
Thompson & Matthews ditch, irrigation from 20, 23, 27
Thompson Bros. ditch, irrigation from... 20, 23
Tongue River, map showing ditches from tributaries of..... 46, 6
plate showing flume in canyon of..... 9
appropriations from..... 45-6
Treichler & Parsons reservoir site, description of 5
Underground channels, loss of water through 49-5
Upper Canyon Creek, appropriations from 3
Upper Phil Kearney ditch, appropriations from 49-5
Water districts, boundaries of 17-18
Water rights, value of..... 13-14
Willow reservoir site, description of..... 3

O

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 24

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

RIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 24

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

WATER RESOURCES

OF THE

STATE OF NEW YORK

PART I

BY

GEORGE W. RAFTER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

CONTENTS.

	Page.
r of transmittal	7
duction	9
ral statements	10
avorable natural conditions	11
rtificial modifications	11
Water storage	12
rie Canal	13
alue of water to industries	14
wnership of water	14
ical conditions	16
ountains and forests	16
emperature and precipitation	18
ocks and stream flow	21
r systems	23
t. Lawrence system	24
Niagara River	24
Genesee River	25
Oswego River	27
Black River	29
Streams flowing into St. Lawrence River	30
ake Champlain system	31
udson River system	33
Mohawk River	35
Schorarie Creek	35
East Canada Creek	36
West Canada Creek	38
Other tributaries of Mohawk River	40
Hoosic River	40
Battenkill River	40
Fish Creek	41
Sacundaga River	42
Schroon River	43
Tributaries south of Mohawk River	43
llegheny River system	44
usquehanna River system	44
elaware River system	46
treams of Long Island	47
able water supply	48
un-off of Niagara River	48
un-off of St. Lawrence River	65
un-off of inland streams of New York	66
Discharge measurements of Eaton and Madison brooks	67
Discharge measurements of Oatka Creek	69
Discharge measurements of Genesee River	70
Discharge measurements of Hemlock Lake	75
Discharge measurements of Skaneateles Lake	77

Available water supply—Continued. Page.
Run-off of inland streams of New York—Continued.
Discharge measurements of Hudson River 79
Discharge measurements of Croton River 82
Maximum and minimum flow of streams in New York 87
Floods in Chemung River 87
Low-water flow of Oatka Creek 90
Low-water flow of Genesee River 90
Low-water flow of Hemlock Lake 92
Low-water flow of Morris Run 93
Low-water flow of West Branch of Canadaway Creek 94
Low-water flow of Skaneateles Lake 96
Low-water flow of Oswego River 96
Low-water flow of Black River 96
Low-water flow of Mohawk River 97
Low-water flow of Hudson River 97
Low-water flow of Croton River 98
Summary of knowledge of low-water flow 98
Index 100

ILLUSTRATIONS.

Page.
PLATE I. Erie Canal at the city of Buffalo 12
II. *A*, Beaver Meadow, near Indian Lake, a typical reservoir site in the Adirondacks; *B*, Bog River in the Adirondacks 16
III. Drainage area of Genesee River 24
IV. *A*, Upper and Middle Falls of Genesee River, at Portage; *B*, Genesee River Canyon below Middle Falls, at Portage 26
V. Middle and Lower Falls of Genesee River at Rochester 28
VI. *A*, Dam at the High Falls of East Canada Creek; *B*, High Falls at Trenton on West Canada Creek 38
VII. Big Falls of Battenkill River 40
VIII. *A*, Hudson River above Luzerne; *B*, Sacundaga River near Luzerne 42
IX. Drainage area of Schroon River 44
X. Canyon of Genesee River between Mount Morris and Portage 70
XI. *A*, Erie Canal aqueduct and south side of Main street bridge, Rochester; *B*, Great flood of 1865 at Rochester, showing lumber lodged against aqueduct bridge 90
XII. Upper Falls of Genesee River at Portage 92
XIII. High Falls of Mohawk River at time of low water 96
FIG. 1. Index map of rivers of New York 2
2. Discharge of Genesee River at Mount Morris, New York, 1893 to 1896 7
3. Flood flow of Genesee River, May 18–24, 1894 7
4. Discharge of Hudson River at Mechanicville, New York, 1888 to 1897 8

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, November 26, 1898.

SIR: I have the honor to transmit herewith a manuscript entitled Water Resources of the State of New York, prepared by Mr. George W. Rafter, and to recommend that it be published in the series of pamphlets upon Water Supply and Irrigation. The data herewith presented were brought together by Mr. Rafter during 1897 and transmitted early in February, 1898. Publication has been somewhat delayed by various obstacles which could not be readily overcome. The data are, however, of general interest and value, not only to the people of New York State, but also to engineers and persons in all parts of the country interested in the development and utilization of the water resources. Particular attention is given to the discussion of floods and low-water flow, these being to a large extent the determining factors in considerations of the utilization of water power.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

HON. CHARLES D. WALCOTT,
Director United States Geological Survey.

WATER RESOURCES OF THE STATE OF NEW YORK, PART I.

By GEORGE W. RAFTER.

INTRODUCTION.

The preeminent position of the State of New York is due almost entirely to her great natural water resources. Reaching from the ocean on the east to the Great Lakes on the west, she has gathered to herself the treasures of the foreign world as well as those of half the Western Continent. Her inland rivers, with their great water powers, have been in the past and will continue to be in the future a perpetual source of wealth. Taking into account the commercial supremacy guaranteed by the Erie Canal, it may be said that the history of the State's progress during the nineteenth century is largely a history of the development of her water resources. It is the purpose of the author in this report to relate briefly not only in what manner these resources have been employed, but to indicate the recent lines of development and the probable future of the State if her water is utilized to the fullest degree. It is proposed to describe in a general way the river systems, giving brief descriptions of several of the more important utilizations of water in New York, together with a discussion of some of the economic problems confronting the people of the State.

As regards the water power of New York, it may be noted that the Tenth Census of the United States, 1880, Vols. XVI and XVII, gives in great detail the statistics of the main water powers as they existed in 1882. Many of these show considerable increase at the present time, although the new works are for the most part similar to those described in the census report, and hence present few additional features of interest. Several of the recent plants, however, are on quite different lines both as to their scope and as to the method of development adopted. It has therefore seemed more important to describe a few of the new plants and to give the main facts of the great storage projects of the Hudson and Genesee rivers than to spend time on small *and relatively unimportant* powers.

The peculiar relation of the State to water-power development on the main rivers of New York should be here mentioned. Owing to the circumstances of the early settlements and the development of the canal system, the State has assumed ownership of the inland waters, or, at any rate, of all streams used as feeders of the canals. This assumption has worked injustice to riparian owners, and is at present a bar in the way of the full development of important streams by private enterprise.

The data embodied in this report have been gathered from many sources—the annual reports of the State engineer and surveyor, the superintendent of public works, the forest commission, the State board of health, the State weather service, and other public documents. The data in the reports on the Water Power of the United States, Tenth Census, have been used in many cases where later data are not available. During the years 1896 and 1897 the author, in addition to his regular duties in the State engineer's department, gathered a large amount of information bearing on the water resources of the State and not published in the annual reports of the State engineer's department. Much of this is in the way of piecing out earlier information and bringing the subject up to date. By the courtesy of the State engineer and surveyor these special data have been embodied in the present report.

The figures as to drainage area have been obtained by checking on French's map, published in 1860, those given in the reports on the water power of the United States, Tenth Census, so far as they are available, and by planimeter measurement on the topographic atlas sheets of the State made by the United States Geological Survey. Bien's Atlas of the State of New York has also been used as a check in some cases, and a number of areas have been taken from the report of the Deep Waterways Commission.

The elevations of points above tide water have been compiled from all available sources of information, such as Dictionary of Altitudes in the United States, Bulletin No. 76 of the United States Geological Survey; the reports of the New York State survey and railway canal profiles; the topographic atlas sheets of the United States Geological Survey, and the reports on the water power of the United States, Tenth Census, 1880.

GENERAL STATEMENT.

A report of this character is prepared for the benefit of two classes: First, professional or business men who read during leisure hours in order to add to their stock of general information; second, engineering specialists, physicists, and men of expert scientific attainments generally who desire full details as part of their stock of professional knowledge. The latter class will naturally study the details, while

for the former a succinct statement of the results of the study is generally sufficient. It is from this point of view that a statement of the general results of the study of the water resources of New York is here introduced at the beginning of the report.

FAVORABLE NATURAL CONDITIONS.

New York State is great in water resources, not only by virtue of her position between the Atlantic Ocean and the Great Lakes, but because topographic, geologic, and climatic conditions have combined to make her the highway of commerce as well as the manufacturing center of the United States. Some of the contributing causes to this preeminent position may be found in her mountain systems, affording three great water centers, from which large streams descend to the neighboring lowlands, affording large opportunities for the economic development of water power. As regards water power, the other chief contributing causes are the possession, as part of her domain, of Niagara and St. Lawrence rivers, with their extensive possibilities for future water-power development.

A study of the climatology of New York shows that in nearly every portion of the State the amount and distribution of the rainfall are such as to insure a large enough run-off of streams to furnish, even under natural conditions, considerable water power.

ARTIFICIAL MODIFICATIONS.

Natural conditions have been largely interfered with by the cutting off of forests and the consequent extensive development of the agricultural interests of the State. As a tentative proposition, it is assumed that the general cutting off of the forests of New York State has decreased the annual run-off of streams issuing from the deforested areas to a depth of from 4 to 6 inches per annum.

The run-off of Niagara River has been commonly assumed, on the authority of the Lake Survey, at about 265,000 cubic feet per second. The recent studies indicate that the extreme low flow of a cycle of minimum years may be not more than 60 per cent of this figure. From this point of view the people of the State of New York have the greatest possible interest in any project which would tend to decrease the low-water run-off of that stream. Such interest is equally pronounced in the case of St. Lawrence River.

Measurements of discharge of a number of the inland streams of New York indicate considerable variations in water yield in different parts of the State. For instance, Genesee River, in the western part, in 1895 gave, with a rainfall of 31 inches, a minimum flow for the year of 6.67 inches. The drainage area of this stream is mostly deforested, whence it results that serious floods are frequent. For

example, from May 19 to 24, 1894, the total discharge was nearly 6,900,000,000 cubic feet, the maximum, which occurred at 3.30 a. m., May 21, being 42,000 cubic feet per second. The drainage area above the point where this flood occurred is 1,070 square miles.

The lowest annual run-off thus far measured in the State of New York is that of the Hemlock Lake drainage area, where, in 1880, the total run-off from an area of 43 square miles was only about 3.35 inches.

Oswego, Mohawk, and Hudson rivers and their tributaries in this State all have large pondage on natural lakes, which, with other conditions, tend to maintain the low-water flow. Croton River presents surface geological conditions which tend to increase its low-water flow. Without going into detail, we may say that these streams will yield a minimum flow of about 0.3 of a cubic foot per square mile per second. Variations from this limit are given in the chapters specially discussing minimum flow.

As a typical flood stream of the State, Chemung River may be mentioned, where serious floods, due to deforestation of a mountainous drainage area, have become so common as to necessitate the carrying out of extensive protection works at the large towns on that stream.

WATER STORAGE.

Large development of water power on Genesee River has led to a demand for extensive storage reservoirs on that stream. The surveys made by the State indicate that a storage reservoir of 15,000,000,000 cubic feet capacity can be constructed at a cost of $2,600,000, or at the rate of about $173 per million cubic feet stored. It is considered that the construction of such a reservoir is commercially feasible, and, provided the State legislature will grant the necessary permission, the project will probably be carried out by a private company. The developed water power of Genesee River has increased from about 6,000 horsepower in 1882 to about 18,000 horsepower in 1898.

Extended studies have also been made of the possibilities of water storage on Hudson River, where the water power has increased from less than 13,000 horsepower in 1882 to something like 55,000 horsepower at the beginning of 1898. The studies, so far as carried, show that it is possible to create on that stream a continuous, permanent power of about 175,000 horsepower. Probably when the studies are complete it will appear that considerably more than this can be developed at a cost which will be commercially feasible.

The great power developments on Niagara River at Niagara Falls, and on St. Lawrence River at Massena, are the most significant industrial movements now taking place in the United States. The future power of these two streams may easily be placed at several hundred thousand horsepower.

ERIE CANAL AT THE CITY OF BUFFALO.

ERIE CANAL.

Erie Canal, a view of which is given on Pl. I, was the first great velopment of the internal water resources of New York, and grew t of the demand for transportation facilities between the Atlantic aboard and the Great Lakes. The impulse which it gave to the velopment of New York State, and of the entire territory tributary the Great Lakes, can hardly be estimated. Taking into account far-reaching consequences, it may be considered the greatest pub- work thus far carried out in the United States. Nevertheless, ie Canal has not only passed its day of usefulness, but, to some tent, stands in the way of future development, the chief cause for s being a too pronounced regard for the canal's former greatness. e historical matter cited in the body of the report may serve to indi- e how strongly the feeling that Erie Canal should be maintained perpetuity has been impressed upon the people of the State of w York.

By way of illustrating the rise and decline of Erie Canal, it may be ed that in 1837 the total freight carried was 1,171,296 tons, valued at ,809,288; in 1880 the total freight carried was 6,457,656 tons, valued $247,844,790; in 1895 the total freight carried was 3,500,314 tons, ued at $97,453,021. Statistics show that the great bulk of all the ight now carried on Erie Canal is through freight carried for estern producers, local business being only a small per cent of the ole. Statistics show that freights are now carried by railways as eaply as they can be carried by the canal, and this, too, at a profit, ile the canal, in order to obtain any freight at all, has been obliged do away with all tolls, thus making the cost of shipment by canal bare cost of transportation proper.

In 1895 an improvement of Erie Canal was authorized at a cost of 000,000. The work of this improvement is now in progress. cently it has been found that the cost will be $16,000,000, instead of 000,000, as originally expected. On this basis, and throwing out the account former expenditures, we may say that Erie Canal will t the people of the State of New York annually at least $1,230,000. suming a traffic for the enlargement of the canal of 5,000,000 tons annum, carried 200 miles, we have a total of 1,000,000,000 ton-miles annum on which the people of the State of New York must pay in way of interest and cost of maintenance and operation about 1.25 lls per ton-mile, while canal freights now average about 1.2 mills ton-mile; hence the people of the State of New York will be liged to pay under the new conditions over 50 per cent of the total t of the transportation. At present the local canal freights are only per cent of the total.

Owing to her extensive inland-navigation system the experience of w York as to loss of water from artificial channels has been very

extensive. Measurements made at different times show that in a canal of the dimensions of the original Erie Canal, which had a section of 40 feet water surface, 28 feet bottom, and 4 feet depth, there should be provided, in order to cover the evaporation and percolation losses, a water supply of about 100 cubic feet per mile per minute. Details relating to this special subject may be found further on.

The State early adopted the policy of leasing surplus waters of Erie Canal for power purposes. The most extensive development growing out of such leases is that at Lockport, where several thousand horsepower are in use supplying establishments valued at about $2,500,000 a year, and employing nearly 1,900 operatives, with an annual product of about $3,000,000.

VALUE OF WATER TO INDUSTRIES.

Water power is extensively sold at Oswego, Cohoes, and Niagara Falls, and to some extent at Rochester. It will also be extensively sold at Massena when the development there is completed.

The value of the internal waters of the State to some of the leading industries, such as the lumber industry and the wood-pulp and paper industry, may be noted. On Hudson River from 1851 to 1897, inclusive, the total number of logs taken to market by water transportation was 23,313,585, these market logs furnishing 4,662,717,000 feet B. M. of lumber. The cost of driving logs from the head waters of the Hudson to the Big Boom above Glens Falls is said to be from 50 to 75 cents per thousand feet B. M.

The wood-pulp and paper industry is developed in New York State to a point beyond that reached in any other State of the Union. On January 1, 1898, there were at least 125,000 net water horsepower in use in the State in the production of mechanical wood pulp, while probably from 30,000 to 35,000 more are consumed in operating paper mills.

One obstacle to the easy operation of water power in this State is the formation on many streams of frazil or anchor ice. A study of the formation of frazil and anchor ice, as made by the Montreal harbor commissioners, indicates that it may be possible to learn in the future how to remedy this difficulty.

OWNERSHIP OF WATER.

The sand areas of Long Island present conditions of water yield different from those of the other drainage areas of the State. We have here an extended region of coarse, deep sand, into which the rainfall sinks easily, there being almost no surface run-off. These sand areas form subterranean reservoirs, from which from 0.7 to 0.8 cubic foot per square mile per second may be drawn, the same as from artificial reservoirs on the earth's surface, these natural underground

reservoirs possessing the advantage of furnishing a filtered water of a high degree of purity.

The taking of the water supply of Brooklyn from the sand areas of Long Island has led to the development of legal principles relating to rights in underground water somewhat different from those derived from the common law of England. The decision in a test case now before the courts is, in effect, that when subterranean water is taken in large quantity for the supply of cities or for manufacturing purposes the party taking it is liable to the adjacent landowners the same as in the case of diverting surface water.

Owing to the development of the internal navigation system, and the consequent assumption on the part of the courts and State officials that the State's rights to the inland waters were, in effect, paramount to all other rights, no general mill act has ever been enacted in the State. Nevertheless, the demand for water storage on the streams of the northern part of the State and on St. Lawrence River, which are not in any way tributary to Erie Canal, has led to the enactment of a number of special laws which have the force of mill acts in that they grant the right of eminent domain for the purpose of improving the hydraulic power of streams. This phase of development of the laws of this State relating to riparian rights is an exceedingly interesting one.

Titles to lands under water and water rights have been considerably complicated in New York because of the peculiar circumstances of the early settlement. It is well-established law that Hudson and Mohawk rivers belong to the State, while the Genesee and the other large rivers belong to the riparian owners. A confusion of ideas arising out of such contradictory facts as these has undoubtedly assisted in obscuring the real relations of the State to riparian owners in New York.

In the case of Black River large quantities of water have been diverted for the supply of Black River and Erie Canal, which has been compensated for in kind by the construction of a system of State reservoirs on the head waters of that stream capable of storing nearly 3,800,000,000 cubic feet. The State has further recognized the rights of riparian owners in the Black River reservoirs by creating a commission of owners and users of water power on Black River to manage the discharge from the reservoirs.

Skaneateles Lake presents a case where the State, having originally appropriated water for the supply of Erie Canal, has later, by act of legislature, allowed the taking of the said water for a municipal supply.

With restrictive legislation repealed and a proper mill act enacted, we may hope ultimately to develop in New York approximately 1,518,000 gross horsepower, worth to the people of the State anywhere from $150,000,000 to $200,000,000 a year, or as much as the present *entire agricultural* product of the State.

PHYSICAL CONDITIONS.

Considering only the broader topographic features, the streams of the State may be pictured as coming from three main water centers. The first of these in importance is the Adirondack area, the peaks of which are over 5,000 feet in height. Next to this are the Catskill Mountains, in the southeastern part of the State, the greatest heights of which are about 4,000 feet; and third are the highlands of the southwestern portion of the State, portions of the Allegheny Plateau. In extent the Adirondack Region is much the largest, and owing to this fact and its higher elevation it is the most important water center of the State. The Catskill Region, from its lower altitude and from changes introduced by cutting off the forests, is much less valuable in yield of water. The issuing streams are flashy and uncertain, pouring down destructive floods in spring and running nearly dry during the summer and fall. It is, without doubt, largely owing to this fact that water-power development has made less progress in this part of the State than in the other power-producing sections. The Allegheny Plateau has also greatly deteriorated in water-yielding capacity on account of deforestation. The serious effect of such decrease is discussed later in connection with the Genesee River storage project.

MOUNTAINS AND FORESTS.

The mountain belt included in the Adirondacks proper—the Bouquet, Schroon, Kayaderosseras, and Luzerne ranges, with the Highlands immediately to the west—has come to be commonly known as the Adirondack Plateau. Describing this broadly, it may be considered as bounded on the east by Lake Champlain, on the west by the valley of Black River, on the north by the farming regions of the St. Lawrence, and on the south by those of the Mohawk Valley. The mountain belt proper, however, occupies only the eastern and southern part of the Adirondack Plateau. Its greatest width is about 40 miles. The mountain ranges, however, are not always distinct; sometimes these are lateral spurs interlocked and sometimes single mountains occupy the space between the ranges, filling the valleys.

From the Adirondack Plateau streams flow to the north, southeast, and west. The principal streams flowing north, east, and west to the St. Lawrence system are Moose, Beaver, Oswegatchie, Grass, Raquette, St. Regis, Salmon, Saranac, Ausable, and Bouquet rivers. The southern streams, which all belong to the Hudson system, are Sacundaga, Indian, Cedar, Opalescent, Boreas, and Schroon rivers, and East Canada and West Canada creeks. All these streams head in lakes, of which the most important, tributary to the St. Lawrence, are Placid, Saranac, St. Regis, Loon, Rainbow, Osgood, Meacham, Massawepie, Cranberry, *Tupper*, Smiths, Albany, Red Horse Chain, Beaver, Brandeth, Bog

A BEAVER MEADOW, NEAR INDIAN LAKE, A TYPICAL RESERVOIR SITE IN THE ADIRONDACKS.

B. BOG RIVER IN THE ADIRONDACKS.

er Chain, Big Moose, Fulton Chain, Woodhull, Bisby, Raquette, . Blue Mountain. Typical views are given on Pl. II.

'ollowing are the principal lakes of the Adirondack Plateau tribu- y to the Hudson system: Pleasant, Piseco, Oxbow, Sacundaga, Elm, rehouse, Honnedaga, West Canada, Wilmurt, Salmon, Spruce, lar, Lewey, Indian, Rock, Chain, Catlin, Rich, Harris, Newcomb, irteenth, Henderson, Sanford, Colden, Boreas, Elk, Paradox, Brant, hroon, and Luzerne.

The great forest of northern New York occupies the central part of e Adirondack Plateau, and deserves notice from its importance as conservator of the streams issuing from that region. According a map accompanying the report of the forest commissioners of New rk for 1891, the outlines of the great forest are substantially as lows: Its eastern boundary coincides quite closely with a line wn through Keene Valley and thence along the valleys of Schroon ver and the upper Hudson; its southern boundary is for the main rt identical with that of Hamilton County and the town of Wilmurt, Herkimer County, although in some places the forest extends a rt distance into Fulton County; its western boundary is the county e between Lewis and Herkimer counties; its northern boundary runs an irregular line from a point near Harrisville, on the Lewis and St. wrence County line, to the Upper Chateaugay Lake, which is situ- d near the line between Franklin and Clinton counties. This ritory contains about 3,590,000 acres, of which 3,280,000 acres are sidered to be covered with dense forests. Within this region there from 1,300 to 1,400 lakes and ponds, while from it the eighteen portant streams just enumerated diverge in every direction. The eral elevation of the Adirondack Plateau is about 2,000 feet above level of the sea. Little discussion is needed, therefore, to show the at value of this elevated forest-covered plateau as a conservator of natural waters of the State.

ne important utilization of the waters of this State formerly was carrying of logs to market through the various streams. By son of the clearing off of the forests, that business has gradually lined, until, except in the Adirondack Plateau, it is now of e importance. It has been the policy of the State for a number years to acquire, as far as possible, by tax title and purchase, ies of land in the Adirondack forest for the purpose not only conserving the forests in order to increase the yield of streams, for the further purpose of creating a forest park worthy of the at Commonwealth of New York. In order to carry out this project forest-preserve board has been empowered to purchase lands hin the forest, or, failing to agree on terms with the landowners, ake lands under condemnation proceeding.[1]

---

he State holdings in the Adirondack Region up to the year 1895 may be determined by ref- ce to a map of the Adirondack forest and adjoining territory as issued by the fisheries, e, and forest commission in 1895.

The Adirondack Plateau is a rugged, rocky region, sparsely populated, and worthless for agriculture. Its chief value lies in a complete utilization of such natural resources as attach to its unparalleled water-yielding capacity. From this point of view it may easily become an important factor in the future development of New York. To insure this result the water yield of every stream of the region needs to be conserved by reservoir systems.

TEMPERATURE AND PRECIPITATION.

In 1826 an elaborate system of meteorological observations at the State academies was inaugurated by the board of regents of the University of New York and was continued until 1863. During a portion of this period, and later, a large amount of data was obtained by volunteer observers of the Smithsonian Institution and at military posts. In 1871 the United States Signal Service took charge of work of this character, and in 1889 the State meteorological bureau was organized. There is, therefore, a large accumulation of material regarding the local climate, such that an account of it would extend beyond the limits of this paper. For details reference should be made to the volumes on meteorology published by the board of regents of the State University; also to the bulletins and annual reports of the State meteorological bureau and of the United States Weather Bureau.

To facilitate the study of meteorological data the individual stations have been grouped in accordance with topographic features and geographic position, the subdivisions being as shown in the accompanying table. The names indicate in a general way the relative position, the Western Plateau including the portion of the Appalachian Highlands west of Seneca Lake, and the Eastern Plateau the remainder of the Appalachian Highlands from Seneca Lake eastward to the Hudson Valley.

The average annual temperature is generally taken as decreasing with altitude at the ratio of 1° F. to every 300 feet of elevation, the rate being somewhat below this average in winter and above it in summer. An approximate determination for the State indicates that the rates of decrease are .3° F. per hundred feet elevation for the winter, and .4° F. per hundred feet for the summer. For the mountains of northern New York a much smaller variation than .3° F. appears to hold for the winter months.

verage monthly and annual temperatures for the 22-year period, 1871–1892. (a)

	Western Plateau.	Eastern Plateau.	Northern Plateau.	Atlantic coast.	Hudson Valley.	Champlain Valley.	St. Lawrence Valley.	Great Lakes.	Central lakes.	Mohawk Valley.	Average of the ten regions.
ltitude (feet)	1,287	1,070	1,578	82	221	186	431	484	645	639	662
nuary	22.0	21.3	16.0	30.5	21.5	16.3	15.9	23.4	24.2	21.0	21.6
bruary	23.6	22.5	16.8	31.6	26.9	17.5	17.3	24.3	25.8	22.8	22.9
arch	28.5	28.6	24.0	35.9	38.2	25.9	26.5	29.9	30.6	28.8	29.2
pril	41.7	42.0	36.8	46.7	46.2	40.8	40.4	42.0	44.2	42.8	42.3
ay	54.8	55.0	57.8	57.6	58.8	55.2	55.5	54.8	57.3	55.8	55.7
ne	64.3	64.8	60.3	67.0	68.1	64.7	64.2	64.9	66.0	65.0	65.0
ly	68.6	68.6	63.8	72.3	72.0	69.9	68.2	69.8	70.8	69.2	69.3
ugust	66.6	66.3	62.5	71.1	69.6	67.5	65.9	68.1	68.7	67.3	67.4
eptember	59.3	61.5	55.0	65.3	62.8	58.6	58.4	61.1	61.6	60.0	60.4
ctober	47.3	47.2	43.3	55.1	50.8	46.9	45.7	49.3	49.6	48.0	48.3
ovember	35.5	35.6	31.0	43.9	39.0	34.5	33.2	37.3	37.1	35.0	36.3
ecember	26.7	25.9	21.2	34.5	28.5	21.8	22.2	28.4	28.3	25.8	26.3
Average	44.8	44.7	40.3	50.8	48.4	43.2	42.8	46.0	47.0	45.2	45.4

a From fifth annual report of the State weather bureau.

The intimate relation which exists between air circulation and precipitation in New York is one of the most interesting facts to be noted. Owing to lack of moisture in the continental interior, northwest winds in the spring, summer, and fall are essentially dry. In winter their dryness proceeds from low temperature and consequent small vapor-carrying capacity. The winter precipitation is due almost entirely to storm areas passing either actually across or in the vicinity of this State and deriving their supply of vapor from the inflow of moist air which they induce, either from the Atlantic Ocean or from the Gulf Region.

The winter months—December, January, and February—have somewhat less precipitation than either of the other seasons, although in the vicinity of the Atlantic coast, on the southwestern highlands of the State, and in the region of the Great Lakes the winter precipitation is relatively large.

In the spring rising temperature produces a modification and shifting of pressure systems, the winds decreasing in velocity and their directions being more variable than in winter. The frequent showers occurring in April and May appear to be due more than at any other time to the effect of an admixture of air having different temperatures.

In summer the Gulf of Mexico and the Atlantic Ocean contribute large supplies of moisture to northward-moving air currents, and, although cyclonic depressions are less frequent than at any other season, the rainfall accompanying each storm is heavy, and in New York the maximum seasonal precipitation, amounting as an average for the whole State to 10.96 inches, occurs in this season.

As regards the fall months, the rainfall of September is usually light in the region east of the Great Lakes, while in October the maximum general rainfall occurs. As regards meteorological conditions, winter may be considered as beginning in November.

Average precipitation in New York State, in inches.

	Western Plateau.	Eastern Plateau.	Northern Plateau.	Coast region.	Hudson Valley.	Champlain Valley.	St. Lawrence Valley.
Altitude (feet) *a*	1,307	1,056	973	132	230	262	414
January	2.52	2.52	3.11	3.47	2.89	1.73	2.19
February	2.23	2.34	2.78	3.22	2.26	1.35	2.15
March	2.51	2.46	3.06	3.74	2.88	1.94	2.49
April	2.68	2.80	2.66	3.50	2.82	1.88	2.21
May	3.36	3.54	3.45	3.90	3.53	2.63	2.82
June	4.23	4.16	3.28	3.53	3.68	3.16	3.74
July	3.25	4.04	4.09	4.20	4.24	3.24	3.39
August	3.13	3.50	3.50	4.54	3.69	3.39	2.75
September	2.90	3.13	3.19	3.59	2.90	3.09	3.26
October	3.28	3.31	3.47	3.93	3.52	3.12	3.44
November	2.76	2.81	3.48	3.87	3.15	2.61	2.71
December	2.73	2.82	2.90	3.44	2.89	1.92	2.57
Annual	35.58	37.43	38.97	44.93	38.46	30.06	33.52
Storage period	16.03	16.49	17.96	21.29	17.27	11.44	14.43
Growing period	10.61	11.69	10.87	12.27	11.62	9.79	9.68
Replenishing period	8.94	9.26	10.14	11.37	9.57	8.83	9.41

a Average altitude of stations considered.

A study of the data shows that there are a number of contending forces which are distinctively operative in New York, and which by modifying one another tend to produce numerous irregularities of the rainfall. So irregular indeed is the precipitation that frequently places only a short distance apart show wide variations.

In a general way it may be said that the amounts of annual rainfall in different sections of New York are mainly determined by proximity to sources of vapor or to vapor-laden air currents, and by the *character* of the local topography. As regards the latter statement, a more definite form would be that under similar conditions the pre

:ation is in some degree proportionate to the altitude. This rule, le generally true, does not apply to the valley of Hudson River, re the upper portion, including the Champlain Valley, receives a ewhat deficient rainfall as compared with the State as a whole. the west, the Adirondack Plateau receives a marked increase of ıfall, while farther northwest there is a decrease in the valley of St. Lawrence. This is also true of the elevated region in the nity of Hemlock Lake, which, although several hundred feet ıer, has a rainfall considerably less than that at Rochester.

ı the southeastern portion of the State the ocean winds find no truction along the coast, but, passing inland and meeting the upt ranges of the southeastern counties, give a copious rainfall, as ıpared with that of the intervening regions.

Vestern New York, on account of the frequent southwesterly direc- ı of the winds, receives an appreciable portion of its vapor supply n the Gulf of Mexico. The rainfall in central New York, although than that of the southeastern and southwestern highlands, is erally abundant. The principal valleys of the Susquehanna sys- ı, and also the depression of the central lakes tributary to Oswego 'er, show a deficiency as compared with the average of the State.

ı knowledge of the snowfall is important in a study of the water ɔurces, because by reason of the snow lying on the ground con- ıously for several months it is a great source of loss in open regions ject to severe winds, the evaporative effect of the winds tending arry away large quantities of moisture which would otherwise be ilable to maintain stream flow. Thus far the only data relating lepth of snow are those derived from the reports of the State eorological bureau. The following are a few figures so derived: he winter of 1891–92 the total depth of snow at Humphrey, in the stern Plateau, was 119.8 inches; in 1890–91 the total depth at perstown, in the Eastern Plateau, was 110 inches; in 1891–92 the l depth at Constableville, in the Northern Plateau, was 170.7 ıes; in the winter of 1890–91, at Utica, in the Mohawk Valley, the l depth was 165 inches, and in 1891–92, at the same place, 151.6 ıes. The records show that at the places where these large snow- s occurred the ground was continuously covered with snow for eral months. If the winds were of high velocity at the same time evaporation loss must have been very great.

ROCKS AND STREAM FLOW.

mong the principal factors affecting stream flow should be noted structure and texture of the rocks, especially those of the surface. example, in regions with stiff, heavy, compact soils a much larger ɔortion of the rainfall runs off on the surface, passing immediately the streams, than is the case in regions with open, porous soils or *ınsive sandy areas.* A general knowledge of the surface geology

is therefore desirable in a study of the water resources of the State. The relative position and area of the different geologic formations are best shown on the large geologic map of New York prepared under the direction of James Hall, State geologist, by W. J. McGee, and printed by the United States Geological Survey in 1894 (scale, about 5 miles to the inch). A similar but smaller map showing essentially the same features was also printed in the same year under authority of the regents of the university to accompany the report on the mineral exhibit of New York at the World's Columbian Exposition, this being on the scale of approximately 14 miles to an inch. On examining either of these maps one will note the preponderance, so far as area is concerned, of two classes of rocks—the ancient crystallines, which cover a large area in the northern part of the State, and the conglomerates, sandstones, and shales of the Devonian, which form the greater part of the Appalachian Plateau, stretching from Lake Erie across the State to within a short distance of Hudson River, this being the area classified by the State weather bureau as the Eastern and Western plateaus. The streams from the northern crystalline area undoubtedly furnish the best water supply of the State. This may not be due wholly to the character of the rocks, as many other factors contribute to this result.

The sandstones of the Upper Devonian along the northern boundary of Pennsylvania are bounded on the north by the long narrow belts of outcrop of the underlying rocks stretching in a general easterly and westerly direction. The streams pursuing a general northerly course pass in succession across these. As a rule, the soils of the region are heavy, with considerable clay, and the rainfall being absorbed somewhat slowly, a considerable portion of it flows directly into the water courses. The primeval forest has for the most part been cut away and heavy floods are common, such as those of the Genesee and Chemung rivers, described more fully on a later page.

The only streams of this region on which extensive discharge measurements have been made are Genesee River and its tributary, Oatka Creek. Streams of similar character in western Pennsylvania, however, have been measured for a number of years by the Philadelphia water department, and the results of these measurements are available for comparison and discussion. The results obtained on the Pennsylvania streams, the Neshaminy, Tohickon, and Perkiomen, are applicable particularly in estimates of the flow of the tributaries of Delaware River, rising in New York State, and to the more easterly streams which form the Susquehanna.

The drainage basins of the Oswego, Mohawk, and Hudson rivers are so highly composite as regards geologic formations and embrace such a wide variation in topography and surface geology that no definite deductions concerning the effect of the formations on water flow have been drawn. The streams of Long Island, rising among the sands

lls, and gravels of comparatively recent, unconsolidated formations, fer peculiar conditions, which are discussed on a later page.

RIVER SYSTEMS.

The rivers of the State may be classified into seven general systems, hose relative position is shown by the accompanying index map, g. 1. These are:

(1) St. Lawrence system, which includes all waters draining to Lakes rie and Ontario, and Niagara and St. Lawrence rivers.

FIG. 1.—Index map of rivers of New York.

(2) Champlain system, including all streams in the State tributary o Lakes Champlain and George. The Champlain system is in reality subdivision of the St. Lawrence, but made separate here merely for onvenience in discussing the river systems of the State.

(3) Hudson River system, including all streams tributary to the Iudson and its main branch, the Mohawk.

(4) Allegheny River system.

(5) Susquehanna River system.

(6) *Delaware River system.*

(7) The streams of Long Island tributary to Long Island Sound ar the Atlantic Ocean.

The head waters of the branches of Housatonic River in Connec cut flow out of the State to the east, while the head waters of Rama River, in Rockland County, flow from New York into New Jerse These latter are of possible future importance by reason of the nec sity of water for the supply either of Greater New York or, in t case of Ramapo River, also for the municipalities of northern Ne Jersey. Chateaugay River, a tributary of the St. Lawrence, also flows northward into the Dominion of Canada.

ST. LAWRENCE RIVER SYSTEM.

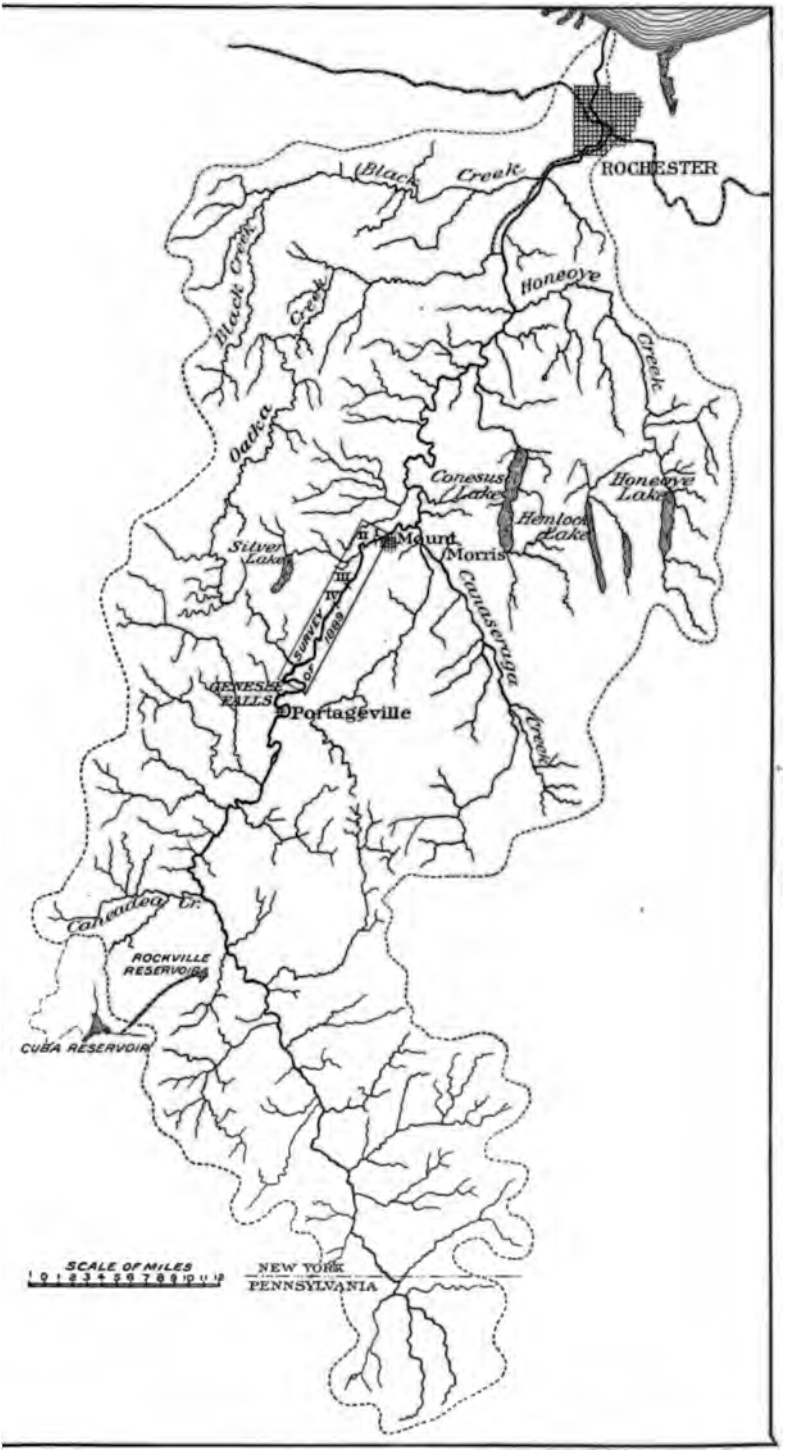
This group embraces the streams tributary to Lake Erie, Niagara River, Lake Ontario, and St. Lawrence River. On the extreme southwest, in Chautauqua County, the watershed line approaches within a few miles of Lake Erie, but at an elevation of several hundred feet above, and as a consequence the streams are short and rapid. A small amount of power is developed on Chautauqua Creek at Westfield, and on Canadaway Creek near Fredonia. Cattaraugus, Buffalo, Tonawanda, and Oak Orchard creeks may also be mentioned as tributaries of Lakes Erie and Ontario and Niagara River in western New York. Buffalo Creek is important as forming a large portion of Buffalo Harbor at its mouth. Tonawanda Creek, which flows into Niagara River at Tonawanda, is used for several miles as a part of Erie Canal. This stream is sluggish throughout nearly its whole course and affords only a small amount of power. The water suppl of the village of Attica is taken from its head waters.

NIAGARA RIVER.

Niagara River forms a portion of the boundary between the Domir ion of Canada and the State of New York. The difference in elevatio between Lakes Erie and Ontario is, approximately, 336 feet, of whic about 160 feet are at Niagara Falls. Between Lake Erie and Niagar Falls the river divides into two channels around Grand Island, whic is 10 miles long and 4 or 5 miles wide. The general course of tl river is from south to north, but in passing around Grand Island tl eastern channel bends westward, and for 3 miles from the foot the island the course of the river is west.

Goat Island lies at the foot of this westerly stretch. On the Ne York side the American channel finds its way around the island the American Falls, which break over the rough ledge at right angl to the main river. The Horseshoe Falls, on the Canadian side, a about 3,000 feet higher up and lie between the west end of Goat Isla and the Canadian shore. At the Canadian Falls the main river aga turns to the north and pursues that general course to Lake Ontar

The elevation of the water surface at the head of the rapids abo the falls is 560 feet above tide water, thus giving a fall from the La



DRAINAGE AREA OF THE GENESEE RIVER.

Erie level to that point of from 12 to 13 feet, of which from 4 to 5 feet are included in the rapids at the city of Buffalo, in front of and just below Fort Porter. The descent in the river from the head of the rapids to the brink of the falls is about 50 feet. At the narrows, half a mile above the whirlpool, the elevation of the water surface is 300 feet, while that of the surface of the still water opposite Lewiston is 249 feet; the fall in this section, which is from 4 to 4.5 miles in length, may therefore be taken at 51 feet, while from Lewiston to the mouth at Fort Niagara the fall is only 2 feet in a distance of 7 miles. The total length of Niagara River is about 37 miles.

On account of the immense water-power developments now taking place at Niagara Falls the run-off of Niagara River must necessarily receive extended discussion in a complete account of the water resources of New York.

GENESEE RIVER.

Genesee River, as shown on Pl. III, issues from the highlands of the Allegheny Plateau in Potter County, Pennsylvania, a few miles south of the New York State boundary. Entering Allegany County, it first runs northwesterly for upward of 30 miles to near the village of Canadea, at which point it turns northeasterly, this direction being generally maintained to the mouth. It flows entirely across the county of Allegany and then for several miles forms the boundary between Livingston and Wyoming counties, after which it crosses the northeast part of Livingston into Monroe County, through which it continues to its mouth at Charlotte. Above Portage its course from the State line is chiefly through an alluvial valley.

From Portage to Mount Morris the river flows through a deep and in some places narrow canyon for a distance of over 20 miles. The Portage Falls, with a total descent including the intervening rapids of about 330 feet, are at the head of this canyon. The Upper Portage Falls have a descent, including the rapids, of about 70 feet. Half a mile below are the Middle Falls, shown on Pl. IV, with a descent of 110 feet; while 2 miles below begin the Lower Falls, consisting of a series of rapids about half a mile long with an aggregate fall of 150 feet. These three falls may be taken as aggregating about 270 feet, exclusive of the rapids. At present no power developments exist. Formerly a sawmill was located at the Middle Falls, but on account of the extinction of the lumber business on the stream it has not been operated for many years.

At Mount Morris, Genesee River issues into a broad, level, alluvial valley from 1 to 2 miles wide, which continues to near Rochester, where there is a descent of 263 feet in about 3 miles. The Upper Falls at Rochester, 90 feet in height, are a cataract in the Niagara limestone, while at the Lower Falls, 94 feet in height, shown on Pl. V, *the Medina sandstone* appears.

The principal tributaries of Genesee River are Canaseraga, Hone-

oye, and Conesus creeks from the east, and Oatka, Black, and Wiscoy creeks from the west. Honeoye, Canadice, and Hemlock lakes are tributary to the Honeoye Creek, and Conesus Lake to Conesus Creek. Silver Lake is another small body of water in the Genesee Basin and tributary to the river by the Silver Lake outlet. Canaseraga Creek joins Genesee River near Mount Morris. From Dansville to its mouth, a distance of 16 miles, this creek flows through a broad alluvial valley with very little fall. Above Dansville the stream is more rapid, but the comparatively small, deforested drainage area limits its value for water power. Honeoye Creek, which is the outlet of Honeoye, Canadice, and Hemlock lakes, furnishes some water power. There are also several mills on the outlet of Conesus Lake.

Formerly there were a number of mills on the Silver Lake outlet, but changed business conditions have led to their decay. The other tributaries of the Genesee have little significance as mill streams. It appears, then, that the two places of importance on Genesee River, from the water-power point of view, are Portage and Rochester. These will be discussed in detail farther on.

The following table gives the detail of the several subdivisions of the drainage area of Genesee River:

Drainage areas, in square miles, of tributaries of Genesee River.

Creek.	Drainage area.	Area above mouth.	Area below mouth.
Cryder	43.3	99.9	143.2
Chenunda	30.0	181.0	211.0
Dykes	68.3	214.0	282.3
Vandemarck	21.6	301.3	322.9
Knights	22.3	323.9	346.2
Phillips	32.3	372.8	405.1
Van Campens	55.7	410.4	466.1
Angelica	82.1	481.1	563.2
White	15.9	569.2	585.1
Black	31.1	595.5	626.6
Crawford	11.8	637.6	649.4
Canaadea	63.3	651.0	714.3
Cold	41.0	745.3	786.3
Rush	35.3	787.0	822.3
Wiscoy	108.6	833.6	942.2
Wolf	19.3	974.9	994.2
Silver Lake	30.1	1,029.2	1,059.6
Coshaqua	82.0	1,059.6	1,141.6
Canaseraga	258.7	1,148.4	1,407.1
Beards	41.3	1,423.1	1,464.4
Conesus Lake	88.8	1,555.5	1,643.9
Honeoye	262.6	1,675.9	1,938.5
Allens	198.1	1,947.1	2,145.2
Black	211.8	2,168.5	2,380.0

The total drainage area of Genesee River at its mouth is 2,45[illegible] square miles.

A. UPPER AND MIDDLE FALLS OF GENESEE RIVER AT PORTAGE.

B. GENESEE RIVER CANYON BELOW MIDDLE FALLS AT PORTAGE

The following tabulation gives the elevation of Genesee River at various points.

Elevation above tide water of Genesee River at various points.

	Feet.
Mean surface of Lake Ontario	247
Crest of the feeder dam in south part of the city of Rochester	510
Low-water surface of river at New York, Lake Erie and Western Railway bridge near Avon	538
Crest of old Mount Morris power dam	605
Water surface just above Upper Falls at Portage	1,080
Water surface at New York, Lake Erie and Western Railway bridge near Belvidere	1,333

The extreme head waters in Potter County, Pennsylvania, are stated to be considerably over 2,000 feet above tide.

OSWEGO RIVER.

Oswego River flows into Lake Ontario at the city of Oswego. Its basin includes the more important of the inland lakes of western New York. Taking the lakes of the Oswego River Basin in order from west to east, their names, elevations above tide, area of water surface, and tributary drainage area at the foot of each lake are as follows:

Elevation and area of lakes of the Oswego River Basin.

Lake.	Elevation above tide.	Area.	Drainage area.
	Feet.	*Sq. miles.*	*Sq. miles.*
Canandaigua	687.0	18.6	175.0
Keuka	720.0	20.3	187.0
Seneca	443.0	66.0	707.0
Cayuga	380.0	66.8	1,593.0
Oswaco	710.0	12.4	208.0
Skaneateles	867.0	12.8	73.0
Otisco	784.0	3.0	41.0
Onondaga	364.0	4.0	267.0
Cazenovia	900.0	2.8	9.0
Oneida	370.0	80.9	1,300.0

The following are the drainage areas of Oswego River and its principal tributaries:

Drainage areas of Oswego River and principal tributaries.

	Sq. miles.
Oswego River at mouth	5,013
Below junction of Seneca and Oneida rivers	4,868
Oneida River	1,420
Seneca River	3

The subdivisions of the dainage area of Seneca River are as follows:

Subdivisions of drainage area of Seneca River.

	Sq. miles
At junction with Oneida River	3,450
At Baldwinsville	3,136
At Montezuma	2,47[illegible]
Below Cayuga Lake	1,503
At entrance to Cayuga Lake	780
At Seneca Falls	771
Waterloo	745
At foot of Seneca Lake	707
Keuka Lake Outlet	213
Catherines Creek	91

The drainage areas of Cayuga Lake and its tributaries are as follows:

Drainage areas of Cayuga Lake and tributaries.

	Sq. miles
At outlet	813
Cayuga Inlet, including Cascadilla Creek	173
Fall Creek, not including Cascadilla Creek	152
Salmon Creek	90
Taughanic Creek	60

Clyde River, a tributary of Seneca River, is formed by the junction of Canandaigua Outlet and Mud Creek. The latter stream rises in the southern part of Ontario County and flows first north and then east, uniting with Canandaigua Outlet at Lyons. Clyde River joins Seneca River at Montezuma. The following are the drainage areas of Clyde River and tributaries:

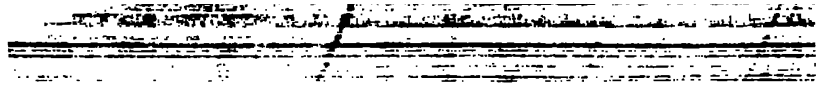
Drainage areas of Clyde River and tributaries.

	Sq. miles
At mouth	889
At Clyde	807
At Lyons, at junction of Canandaigua Outlet and Mud C[illegible]	739
Mud Creek at Lyons	288
Canandaigua Outlet at junction with Mud Creek	451
Canand[illegible]tlet at Phelps	300
Canand[illegible] at foot	175
Canand[illegible]	85

Owas[illegible]ischarges into Seneca [illegible] an outlet 13 miles i[illegible] The following are th[illegible] of Owasco Outlet:

[illegible]inage areas of Oc[illegible]

At mouth	[illegible]
At Auburn	[illegible]
Owasco Lake	[illegible]
Owasco Inlet	[illegible]



The drainage areas of Oneida River and its principal tributaries re as follows:

Drainage areas of Oneida River and principal tributaries.

	Sq. miles.
neida River at mouth	1,420
neida Lake at foot	1,300
ish Creek	480
hittenango Creek, including Cazenovia Lake	306
neida Creek	128

The Oswego River and its tributaries furnish a large number of 'ater powers, the detail of which may be obtained from the Report n Water Power of the United States, Tenth Census of the United tates, 1880. Very little can be added to the statements in that report.

The next stream of any importance tributary to Lake Ontario is ig Salmon River, which rises in the highlands of Lewis County and ows westerly into Lake Ontario. This stream was extensively conidered several years ago as the source of water supply for the city of yracuse, the water to be taken at a point about 40 miles distant from he city. Its watershed above the proposed point of diversion comrises 70 square miles of forest land at an elevation of from 1,000 to ,500 feet above tide water.

BLACK RIVER.

Between Big Salmon River and the mouth of Black River there are number of small streams flowing into Lake Ontario, none of which re of special importance. We may therefore pass to a brief descripion of Black River. This stream rises in the western part of Hamlton County and pursues a southwesterly direction, passing across Herkimer County into Oneida County; it then bends to somewhat vest of north through Lewis County, but soon after passing the northvesterly boundary of that county it changes to a general westerly ourse, flowing into Black River Bay at the extreme eastern end of Lake Ontario. Extensive water-power developments are in use on his stream and its tributaries, at Watertown, Lyons Falls, Carthage, Black River, Brownville, Dexter, and other points. There are also a number of State reservoirs on the head waters which will be discussed n detail later. The following gives the elevation in feet of the main oints on Black River above tide water, according to the best availble information.

Altitude of points along Black River.

	Feet.
At mouth	247
Watertown, west line of city	370
Watertown at head of falls	492
Carthage at foot of rapids	669
Carthage at crest of State dam	724

Altitude of points along Black River—Continued.

	Feet.
Lyons Falls at foot	738
Lyons Falls, crest of State dam	802
Forestport, crest of State dam	1,120
North Branch reservoir	1,821
Chub Lake	1,590
Woodhull reservoir	1,854
South Branch reservoir	2,019
Moose River at mouth	803
First Lake, Fulton Chain	1,684
Second Lake	1,684
Third Lake	1,685
Fourth Lake	1,687
Fifth Lake	1,691
Sixth Lake	1,700
Seventh Lake	1,703
Eighth Lake	1,803
Little Moose Lake	1,772
Big Moose Lake	1,787
Beaver River at mouth	724
Beaver Lake at Number Four	1,436

The drainage areas of Black River and its tributaries, in square miles, are as follows:

Drainage areas of Black River and tributaries.

	Sq. miles.
Black River at mouth	1,860
At Watertown	1,820
At Carthage	1,741
At Lyons Falls below Moose River	810
Forestport	275
Above mouth of Sawmill Creek	174
Deer River	107
Beaver River	365
Lovell Creek	34
Independence Creek	90
Martins Creek	29
Otter Creek	60
Moose River	349
Sugar River	66

STREAMS FLOWING INTO ST. LAWRENCE RIVER.

Proceeding along St. Lawrence River we find a number of streams, such as the Oswegatchie, which flows into the St. Lawrence at Ogdensburg; the Grass, which enters the St. Lawrence near the north line of the State; the Raquette and St. Regis, flowing into the St. Lawrence a short distance below the Grass, and finally the Chateaugay, which flows from this State into the Dominion of Canada and thence into the St. Lawrence. These streams all head in and about the Adirondack Plateau and, as a rule, fall rapidly from their sources to near their mouths, affording large water powers, which thus far have

been chiefly utilized for pulp grinding, paper making, and sawing lumber.

There is a lack of definite information in regard to all the streams of the northern part of the State. No detailed surveys of this region have been made. Partial reservoir systems have been constructed on Oswegatchie, Grass, and Raquette rivers. Some of the economic questions involved in the construction of these reservoirs will be discussed in Part II of this paper, Water-Supply Paper No. 25.

So far as can be learned, no measurements have been made of any of the streams tributary to St. Lawrence River proper. It is probable, however, that they are the best water-yielding streams of the State, because they flow from the great northern forest, and because their head waters are in the extensive lake region which lies immediately west of the main Adirondack Mountains, and which extends westward from the base of the main range to the borders of the forest, a distance of nearly 50 miles. This portion of the Adirondack Plateau is comparatively level. As regards geographic distribution, these lakes are most numerous in the northern parts of Herkimer and Hamilton counties and the southern parts of St. Lawrence and Franklin counties. Those in Herkimer County flow into Moose and Beaver rivers, tributaries of Black River. The following are the elevations of a few of the more important lakes of Hamilton, St. Lawrence, and Franklin counties, which are tributary to streams flowing northward into the St. Lawrence:

Elevations of important lakes of Hamilton, St. Lawrence, and Franklin counties.

Lake.	Feet.
Cranberry	1,540
Raquette	1,774
Forked	1,753
Long	1,630
Little Tupper	1,728
Big Tupper	1,552

LAKE CHAMPLAIN SYSTEM.

Lake Champlain has a water area of 400 square miles. The area of its watershed in New York State amounts to 2,950 square miles, in Vermont to 4,270 square miles, and in the Province of Quebec to 740 square miles. The total area of watershed, not including water surface, is 7,960 square miles, or the total area of the drainage basin, including water surface, is 8,360 square miles. Lake Champlain is considered as beginning at Whitehall and terminating at St. Johns, on the Richelieu. Its length is 125 miles and its breadth in the

northern portion about 13 miles. The standard low-water elevation is given at 95.03 feet, and standard high water at 103.78 feet, above tide.

The streams tributary to Lake Champlain are Big Chazy, Little Chazy, Saranac, Salmon, Little Ausable, Big Ausable, and Bouquet rivers and the outlet of Lake George. There are also a few small streams of no special importance.

The streams tributary to Lake Champlain are, as a rule, not of great length, but rising, as they nearly all do, in or near the high mountains of the Northern Plateau they have a rapid descent with an abundant fall. For illustration we may refer to the Saranac River, which has, by the county maps of Bien's Atlas, a length of about 55 miles from its mouth to Lower Saranac Lake. The elevation of Lake Champlain above tide water is 101 feet, while that of Lower Saranac Lake is given at 1,539 feet. Hence the fall in 55 miles of river course is 1,438 feet. Middle Saranac Lake lies at an elevation of 1,542 feet and Upper Saranac at 1,557 feet.

In the case of Ausable River we find a distance by the course of the river of about 40 miles from its mouth to Lake Placid, the elevation of that body of water being 1,864 feet above tide, or 1,763 above Lake Champlain; or, taking the distance by that fork of Ausable River which leads to Ausable Lakes, the distance is about 42 miles to Lower Ausable Lake, the elevation of which is 1,961 feet above tide; hence we have a fall in this stream of 1,860 feet in a little over 40 miles. The water power of the several streams tributary to Lake Champlain has been extensively developed at Plattsburg and other points, as indicated by the tabulations relating to pulp, paper, and lumber interests.

The most southerly tributary of Lake Champlain of any importance for water purposes is the outlet of Lake George, which in about 2 miles has a fall of 222 feet. The greater portion of this is concentrated in the first mile from the lake. The elevation of Lake George above tide water is 323 feet. The area of the lake surface is stated at 50 square miles, and the tributary drainage area above the foot of the lake at 238 square miles. In the absence of accurate topographic maps the drainage area of Lake George, like that of most of the other streams considered, can be given only approximately.

As will be shown later, the streams in eastern New York can not be depended on to furnish a natural flow of more that about 0.3 cubic foot per square mile per second as a minimum in a dry year. On account of the large water surface of Lake George in proportion to the drainage area, it is possible, by utilizing the storage on the lake surface, to realize a much larger quantity. From 0.7 to 0.8 cubic foot per second per square mile may be assumed as a conservative estimate, the results being based on allowing the water to flow out of the lake 24 hours per day for only 310 days in the year. On this

basis we may assume a mean flow for minimum dry years of about 200 cubic feet per second. Since the entire 222-foot fall of the Lake George Outlet is now utilized, we may place the permanent power in a dry year at about 5,000 gross horsepower. The village of Ticonderoga, at which this power is all utilized, had a population in 1890 of 2,267.

Wood Creek, the most southerly tributary of Lake Champlain, is of interest in a study of the water resources of New York, chiefly because of its relations to Champlain Canal, its channel being utilized for several miles as part of the canal. At Fort Ann there is considerable power developed on one of its tributaries, used at present for grinding pulp.

HUDSON RIVER SYSTEM.

Hudson River, with its principal tributary, the Mohawk, is the most important river of the State. From its mouth to Troy, a distance of over 150 miles, it is a great inland estuary subject to tidal action, and because of its great length and the large fresh-water inflow it is unique among inland estuaries. From the first landing of the Dutch on Manhattan Island to the present time it has been an important channel of commerce. On his voyage of discovery in 1609, Hendrik Hudson ascended to the head of tide water, and doubtless discerned the possibilities of future settlement which were so soon realized at Albany, Waterford, and Schenectady. The tidal action of Hudson River originally terminated at the rapids above Troy, but its present termination is a few miles below, at the Troy dam, a structure erected about 1820 as a part of the State canal system. There is a lock at the east end of this dam through which canal boats pass into the pool above, thus enabling them to reach Lansingburg on the east side of the river, or Waterford on the west side, where they may enter Champlain Canal.

Below Troy the tributaries of Hudson River are mostly small and generally not of very great importance, although some of them have considerable power development. One of them, Croton River, is the principal source of water supply of the city of New York. On this part of the river the drainage basin is rather narrow, and many of the streams issuing from the highlands at either side have such small drainage areas as to carry only moderate quantities of water. In descending the river from Troy the principal streams are, on the west side, Normankill, Catskill, Esopus, and Rondout creeks, and on the east side Kinderhook, Wappinger, and Fishkill creeks and Croton River. Harlem River, connecting the Hudson with East River, may be mentioned, in view of its value to navigation interests, as an important feature of the water resources of New York.

The following are the elevations of mean tide, mean low tide, and mean high tide above mean sea level at New York, and the mean rise

and fall of tides at various points along the tidal estuary between New York Bay and the Troy dam:

Mean tidal elevations in feet at various points between New York Bay and Troy dam.

Locality.	Mean tide.	Mean low tide	Mean high tide.	Mean rise and fall.
Sandy Hook	.....	.....	.....	4.70
Governors Island	0.00	2.20	2.20	4.40
Dobbs Ferry	0.18	1.62	1.98	3.60
Coxsackie light-house	1.68	0.17	3.53	3.70
New Baltimore	1.73	0.02	3.44	3.42
Colymans	1.88	0.44	3.31	2.87
Castleton	2.09	0.82	3.35	2.53
Van Wies	2.13	0.82	3.29	2.33
Albany	2.43	1.27	3.59	2.32
Nail works	2.78	1.81	3.75	1.94
Troy dam	3.77	3.37	4.17	0.80

The most important tributaries begin above Troy. Ascending on the west side we find Mohawk River, the outlet of Saratoga Lake, Sacundaga River, Stony and North creeks, and Indian and Cedar rivers. On the east side there are Hoosic, Battenkill, Schroon, and Boreas rivers. Above the mouth of Cedar River the main North or Hudson River is considered to extend to and beyond Lake Sanford, including Opalescent River and the streams issuing from the high Adirondacks.

The following gives the height above tide water at New York of a number of points on Hudson River.

Height above tide water at New York of points on Hudson River.

	Feet.
New York (at mouth)	0.0
Troy	3.6
Saratoga dam (crest)	102.0
Fort Edward (below dam)	118.0
Glens Falls (crest of feeder dam)	284.0
Mouth of Sacundaga River	556.0
Mouth of Stony Creek	584.0
Mouth of Schroon River	608.0
At Glen Bridge	728.0
At Riverside Bridge	875.0
At North Creek Bridge	998.0
At North River	1,050.0
Mouth of Boreas River	1,140.0
Mouth of Indian River	1,415.0
Mouth of Cedar River	1,460.0
Lake Sanford	1,733.0

MOHAWK RIVER.

wk River, the most important tributary of the Hudson, rises western-central part of the State, near the Lewis and Oneida line. It flows in a southerly direction to the city of Rome, hich it takes an easterly course across the State, emptying into lson a little above Troy. The principal tributaries are Scho- East Canada, West Canada, and Oriskany creeks.

following are the elevations above tide water of a number of along Mohawk River:

Elevations of points along Mohawk River.

	Feet.
a	12
ohawk Aqueduct	162
ady	214
f Schoharie Creek	270
, above feeder dam	431

are two principal falls of Mohawk River, the Great Falls at and the Little Falls at the city of the same name, where are he only important water powers thus far developed on this At Cohoes are the Great Falls, about 120 feet in height, on he Cohoes Company has developed about 105 feet. At Little ere is a total fall of about 45 feet occurring in a little over half Of this, from 38 to 40 feet are utilized by three dams. Aside small amount of power developed below Cohoes, just above routs" of the Mohawk, there are no water-power developments stream other than those of Cohoes and Little Falls, except a mportant mills on the extreme head waters. The waterworks ity of Rome, at Ridge Mills, 2 miles north of Rome, where a ower pumping system is in use, may, however, be mentioned.

following are the principal subdivisions of the drainage areas wk River:

Subdivisions of drainage area of Mohawk River.

	Sq. miles.
a	3,400
outh of Schoharie Creek	3,100
Falls	1,275
	524
	184

SCHOHARIE CREEK.

arie Creek rises in the southern part of Greene County, whence 18 miles northwesterly and then northerly about 50 miles to awk. The principal subdivisions of the drainage area are as

Subdivisions of drainage area of Schoharie Creek.

	Sq. miles.
h	947
Bridge	884
	808

The drainage area of Schoharie Creek comprises the greater part of Schoharie County and portions of Greene, Albany, Delaware, Otsego, Montgomery, and Schenectady counties. Its head waters drain the western and northern slopes of the Catskill Mountains. At Central Bridge, about 19 miles from the mouth of the creek, the water surface is 560 feet above tide water; at the mouth the elevation is 274 feet. In spite of this large fall Schoharie Creek is not considered especially valuable for water-power development. It is subject to great extremes of flood and low-water flow. This is probably explainable by the nearly complete cutting off of the forests from the drainage area many years ago. The water powers thus far developed in the Schoharie Creek Basin are nearly all small and unimportant.

EAST CANADA CREEK.

The second important tributary of the Mohawk is East Canada Creek, which rises in the southwestern part of Hamilton County and flows southerly, joining the Mohawk at East Creek, about 7 miles from Little Falls. According to a map furnished by Stephen E. Babcock, of Little Falls, the total drainage area of East Canada Creek is 285.7 square miles, of which 58.2 square miles are in Hamilton County, 98.4 square miles in Herkimer County, 128 square miles in Fulton County, and 1.1 square miles in Montgomery County. Following are the elevations of principal points on East Canada Creek:

Elevations of principal points on East Canada Creek.

	Feet.
Bottom of Beardslee Falls near mouth	0
Top of Beardslee Falls	105
Bottom of High Falls	327
Top of High Falls	379
Crest of dam at Dolgeville	445
Mouth of Spruce Creek	477
Mouth of Fish Creek	559
Emmonsburg	646
Stratford	789
Oregon	1,140

The distance from the mouth of the stream to Oregon is about 25 miles.

The principal tributary of East Canada Creek is Fish Creek, which is the outlet of the East Canada lakes. The distance from its point of junction with East Canada Creek to the mouth of the Canada lakes outlet is about 9 miles, and the total rise in this distance 635 feet. The outlet of the lakes, which is nearly level, is about 3.5 miles long. There are no falls of any magnitude on this creek. For the first 5 miles from its mouth Fish Creek rises 245 feet, and from that point to the mouth of the outlet of the East Canada lakes, a distance of 4 miles, the rise is 390 feet.

›cond tributary of East Canada Creek is Spruce Creek, which
:al length from its mouth to its head in the Eaton Millpond
; 8.7 miles, the total rise in this distance being about 550 feet.
›w the Eaton Millpond there is a fall of 180 feet in 2,000 feet.
ıbury Center, Spruce Creek falls 85 feet in about 900 feet.
'om the development at Dolgeville, and small developments
lslee Falls and at one or two other points, very little use has
been made of the water power of East Canada Creek. It is
›, however, that within a few years the water power of this
vill be nearly all utilized.

ling to a manuscript report on the water power of East Can-
›k, by S. E. Babcock, the fall in this stream for the first 1,500
n its junction with Mohawk River is very slight. At this
e first rapids are encountered, where it has been proposed to
a water power, with a head of about 60 to 70 feet. About
1,200 feet farther upstream there is an additional fall of from
feet. This takes us to the top of the so-called Beardslee
ferred to above.

also been proposed to construct an extensive system of power
nent by a series of dams on East Canada Creek, some of the
ıf which may be gathered from the following table:

Plan of power development on East Canada Creek.

Location.	Head (in feet).	Horse-power.	Estimated cost.	Cost per horse-power.
dges	43	1,172	$108,427	$92.51
eet	26	1,023	73,667	72.01
	29	1,141	30,910	27.10
iate	22	865	46,090	53.28
s	72	2,700	56,320	20.86
low High Falls)	74	2,956	125,092	42.40
low High Falls)	34	1,360	56,408	41.40
gham's mill)	44	1,778	135,410	76.16
gham's mill)	44	1,778	129,800	73.00
Falls	105	5,112	128,326	25.10
als and mean	423	19,885	890,450	44.80

›lan of power development further includes the construction of
ɔ of 1,250,000,000 cubic feet, which is estimated to cost $148,000,
a total for the whole development of $1,038,450. With these
›he final cost per net horsepower becomes $52.22. The esti-
ading to this result include cost of land to be flooded, masonry
and head works, turbine water wheels, flumes and head feed-
raceways, waste gates, power stations, racks, engineering

and superintendence, etc. So far as the actual power developments are concerned, the work can probably be constructed for the estimates, but the cost of the storage is, in the author's opinion, somewhat too low. The total number of dams which it is proposed to build is stated at 40, thus giving an average of only $3,700 per dam. This sum would only build timber dams of the most temporary character. The proper operation and repairs of this number of dams, scattered over an area of 200 square miles, would entail in the end an annual expense of $30,000, which is the annual interest at 5 per cent on $600,000. To obtain the real capitalized cost we need then to add $600,000, which gives an amended total of $1,638,450, whence the cost per net horsepower for the entire system would become $82.40.

At present an electric-power station is in process of installation by the Dolgeville Electric Light and Power Company at the high falls just below Dolgeville, shown on Pl. VI, by which it is expected to develop 1,200 net horsepower. The wheels to be set are two twin horizontal 36-inch Victor special wheels, to work under a 72-foot head, and which are claimed by the manufacturers to yield, at full capacity, 600 net horsepower each. A portion of the power generated at this station is to be used at Dolgeville for manufacturing, and the balance, it is stated, will be transmitted to Little Falls, 8 miles distant.

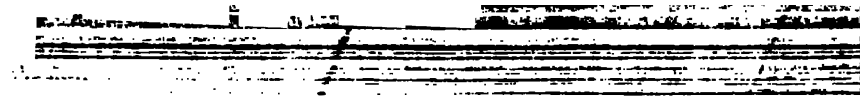
Dolgeville is the seat of the piano-felt and other industries established by Alfred Dolge & Son. The power for the establishments now in operation is derived from two 35-inch Victor turbines, working under a 25-foot head, and rated by the manufacturers to furnish, when running at full capacity, 229 net horsepower each, or a total of 458 horsepower. According to the manufacturer's catalogue, these wheels will consume 197 cubic feet per second when working at full capacity, and the statement is made that they are ordinarily so worked. The drainage area of East Canada Creek above Dolgeville is about 250 square miles; hence the present development is based upon a minimum flow of 0.79 cubic foot per second per square mile. As there is very little pondage at Dolgeville, it may be assumed that the power is sometimes short in a dry season, although the effect of the pondage of the large number of lakes and ponds on the head waters of East Canada Creek will undoubtedly be to increase considerably the minimum flow.

WEST CANADA CREEK.

West Canada Creek, the third important tributary of the Mohawk, rises near the center of Hamilton County and flows southwesterly about 40 miles, by general course, to the eastern edge of the town of Trenton, in Oneida County, where it turns and first runs southeasterly and then southerly for a total distance of 20 miles, finally empty-

A. DAM AT HIGH FALLS OF EAST CANADA CREEK.

B HIGH FALLS AT TRENTON, ON WEST CANADA CREEK.



the Mohawk at the village of Herkimer. The total drainage
ove Herkimer is given as 548 square miles. This creek has
e in the Canada lakes, which are about 40 miles northeast
e village of Prospect. These lakes are known separately as
st, Middle, and East Canada. The principal lake of this
s an elevation of 2,348 feet above tide water. The drainage
he village of Prospect, where there is a natural fall of about
is 375 square miles. At Trenton Falls, 3 miles below, the
escends about 200 feet in half a mile. Ascending the stream,
cipal falls, in order, are: Sherman Falls, 24 feet; High Falls
n Pl. VI), 105 feet; Mill Dam Falls, 14 feet; Suydam Falls, 12

ling to a report made by Wallace C. Johnson, under date
1 17, 1896, to the Trenton Falls Power Company, it appears
he 375 square miles of drainage area above Prospect about
re miles lie at an elevation of between 2,000 and 3,000 feet
de water, the average elevation of this portion being about
t. Of the remaining 200 square miles above Prospect the
elevation is placed at not less than 1,600 feet. The Trenton
wer Company is reported as intending to develop an exten-
age on the head waters of this stream, thus enabling it to pro-
veral thousand electrical horsepower at Trenton Falls for
sion to Utica, Rome, and other towns in the vicinity. Gen-
s have been prepared by Mr. Johnson, but the details of the
re not at hand. Judging from the data at hand, the author
ed to place the minimum flow of West Canada Creek at from
.35 of a cubic foot per square mile per second.

powers are now in use on West Canada Creek at Herkimer,
ille, Newport, and Prospect, as well as at a few points higher up.
s interested in the development of an extensive power project
ton Falls have claimed that a very large storage reservoir
e constructed in the main valley of West Canada Creek a
stance above Prospect, and at very low cost per unit volume
The data are not at hand for accurately determining the cost
rvoir at this place. However, casual inspection of the Rem-
t of the topographic map of the State, made in 1897, shows
h a reservoir would probably be expensive in proportion to
ge gained. A trial estimate shows that with a dam from 80
et in height a storage of about 2,000,000,000 cubic feet may
ned. The cost of the dam necessary to store this quantity of
an hardly be placed as an experimental figure at less than
00, whence the cost per 1,000,000 cubic feet stored would
$500. This approximate estimate has no other significance
indicate the importance of studying large reservoir projects
l before deciding as to their feasibility.

OTHER TRIBUTARIES OF MOHAWK RIVER.

As less important tributaries of the Mohawk, Sauquoit and Oriskany creeks may be mentioned. Sauquoit Creek rises in the southeastern part of Oneida County and runs northerly, emptying into the Mohawk about 2 miles west of Utica. Its drainage area is given as 62 square miles. Oriskany Creek rises in the eastern-southerly portion of Madison County and flows northerly into the Mohawk 6 miles west of Utica. Its drainage area is given as 135 square miles. There is considerable water power developed on both Sauquoit and Oriskany creeks.

HOOSIC RIVER.

The most important tributary of the Hudson from the east is Hoosic River, which rises in the mountains of Berkshire County, Massachusetts. It first runs northwesterly, passing from Massachusetts into the extreme southwestern corner of Vermont and thence into Rensselaer County, in New York. At the northern boundary of Rensselaer County it turns and pursues a westerly course to the Hudson opposite the village of Stillwater. Its drainage area at the mouth is taken at 730 square miles. Its principal tributaries are Little Hoosic River, Walloomsac River, and Tomhannock Creek. The country drained is mainly mountainous, the summits attaining an elevation of from 1,000 to 2,000 feet above tide. The principal water powers developed on Hoosic River, in New York, are at Schaghticoke and Hoosic Falls, with a few at intermediate points. At Schaghticoke there is from 97 to 98 feet fall, broken into falls of 8, 7.5, 24.5, 34.5, and 23 feet. The available statements as to the power at Hoosic Falls are so conflicting that it is thought best to omit them.

Hoosic River is of considerable interest to persons concerned in water-power development on the Hudson below its mouth, because there are two reservoirs on its headwaters which have been constructed by manufacturers in Massachusetts in order to maintain a more equable summer flow. The first of these is the Clarksburg reservoir, on the North Branch of Hoosic River, and at a distance of about 2½ miles above North Adams. The second reservoir is on the South Branch, and is known as the Cheshire reservoir, being situated in the town of that name. The Clarksburg reservoir is stated to flow 156 acres and to have a depth of 22 feet. The Cheshire reservoir flows about 650 acres and can be drawn down about 8 feet. Both these reservoirs are controlled by an association of mill owners on the Hoosic and its branches in the State of Massachusetts.

BATTENKILL RIVER.

Battenkill River, another important tributary of the Hudson, rises in the southwestern part of Vermont, in Bennington County. It first

BIG FALLS OF BATTENKILL RIVER.

ws southwesterly and then westerly irregularly across Washington unty, New York, to the Hudson at a point about a mile above huylerville. The drainage area is taken at 460 square miles. The evation above tide at the mouth of the river is 82 feet, and at the elaware and Hudson Railway crossing, a little south of Shushan, e elevation is 437 feet. This gives a descent of 355 feet in 22 miles, out one-half of which is concentrated within the last 4 or 5 miles of e river's course.

The following is a brief statement of the water powers on the lower ction of the Battenkill, in ascending order from the mouth:

Water powers on the lower section of the Battenkill.

At Clark Mills, the American Woodboard Company, 24 feet head.

At Big Falls (shown on Pl. VII), the dam at the head of the falls gives 106 feet ad, divided into Bennington Falls Pulp Company, 32 feet; Ondawa Pulp and per Company, 30 feet; not utilized, 44 feet.

t Middle Falls, the dam at the head of the falls gives 55 feet head. Here there a leather-board mill, shank mill, sawmill, plaster mill, gristmill, and electric-ht station.

t Greenwich, Dunbar, McMaster & Co., 8 feet head; Palmer's lower dam, 9 t head, furnishes power for gristmill, paint works, shirt manufacturing, scale nufacturing, and plow works; Palmer's upper dam, 6 feet head, furnishes wer for a cotton mill and a paper mill.

t Center Falls, Angel & Langdon Paper Mill, 25 feet head.

t Battenville, Phœnix Paper Company, 10 feet head.

t Rexleigh there is a cotton mill with 6 feet head; at Shushan a gristmill, shirt tory, electric-light station, and foundry, all receiving power from about 14 t head.

In addition to the foregoing there are stated to be undeveloped ater powers on the Battenkill as follows: Between Clark Mills and g Falls, 27 feet; between Greenwich and Center Falls, 8 feet; be-een Center Falls and Battenkill, 10 feet.

It is stated that the utilized powers on the Battenkill are developed to about 30 horsepower per foot fall. They are, however, sometimes rt of water in dry weather. With a drainage area of 460 square les, a minimum flow of 0.3 of a cubic foot per square mile per sec-d would give only about 15.3 gross horsepower per foot of fall. It nferred, therefore, that the Battenkill is an exceedingly good water lder, although definite data derived from stream measurements are tirely lacking.

FISH CREEK.

Fish Creek, a stream tributary to the Hudson at Schuylerville, is outlet of Saratoga Lake. Its chief tributary is the Kayaderosseras eek, which drains the central part of Saratoga County. The drain-area of Fish Creek at its junction with the Hudson is estimated at square miles. Both Fish Creek and Kayaderosseras Creek are ensively utilized for water power.

SACUNDAGA RIVER.

Sacundaga River, a view of which is given on Pl. VIII, is the next important tributary of the Hudson in the ascending order. It has three principal branches, which unite to form the main river in the southeastern part of Hamilton County. The West Branch is the outlet of Piseco Lake; the Middle Branch is the outlet of Sacundaga and Pleasant lakes, while the East Branch issues from a series of small ponds and lakes in the southwestern part of Warren County, not far from Bakers Mills. The East and Middle branches unite a few miles to the north of Wellstown, and West Branch joins a few miles south of Wellstown. The river then flows southeasterly to about 5 miles below Northville, where it turns rather more than a right angle and flows irregularly northeast to the main Hudson at Hadley. The principal tributary of the Sacundaga, aside from its several branches, is East Stony Creek.

The following are the several subdivisions of the drainage area of Sacundaga River:

Subdivisions of the drainage area of Sacundaga River.

	Sq. miles.
Total drainage area at mouth	1,040
South Branch	240
Middle Branch	115
East Branch	124
Stony Creek	212
Main river below Stony Creek	[illegible]

The following are elevations above tide at a number of principal points:

Elevations above tide of points along Sacundaga River.

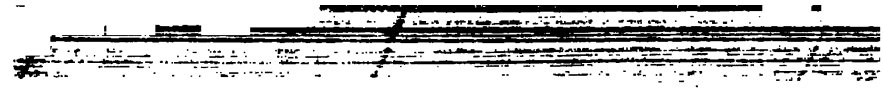
	Feet.
At mouth of river	556
Above dam at Conklinville	697
Northville	[illegible]
Hope Center	[illegible]
Wellstown	[illegible]
East Branch at old tannery	[illegible]
East Branch at foot of High Falls	1,[illegible]
East Branch at head of High Falls	1,[illegible]
East Branch at Brighams Pond	1,[illegible]
Piseco Lake	1,[illegible]
Lake Pleasant	1,[illegible]
Sacundaga Lake	1,[illegible]

From Conklinville to the mouth of the river, a distance of a little over 5 miles, the river falls 141 feet. At present this section of the river is entirely unutilized except by two powers, one at Conklinville and the other about 2 miles from Hadley.

Thus far there are no detailed measurements of the Sacundaga, but since the drainage area is still largely in primeval forest it is without doubt an excellent water yielder.

A. HUDSON RIVER ABOVE LUZERNE

B. SACUNDAGA RIVER NEAR LUZERNE.



SCHROON RIVER.

Schroon River rises in Essex County, along the southern slopes of the highest mountains of the Adirondack group. As shown by the map (Pl. IX), it flows in a general southerly direction for about 45 miles, through Essex and Warren counties, and joins the Hudson just above Thurman. On the boundary between Essex and Warren counties the river flows through Schroon Lake, a body of water nearly 9 miles long and from a little less than 0.5 to 1.5 miles in width.

The following are some of the important subdivisions of the drainage area of Schroon River:

Subdivisions of the drainage area of Schroon River.

	Square miles
At mouth	550
Warrensburg	535
Tumblehead Falls	502
Foot of Schroon Lake	479

Some of the elevations on Schroon River are as follows:

Elevations on Schroon River.

	Feet.
At mouth	610
Schroon Lake	807
Paradox Lake	820
Schroon Falls	840
Elk Lake	1,986

There is no developed water power on Schroon River except that at Warrensburg.

TRIBUTARIES SOUTH OF MOHAWK RIVER.

Kinderhook Creek and Croton River are the chief tributaries of the Hudson south of the Mohawk requiring special mention at this time. Brief consideration of the run-off of Croton River will be given further on. The Columbia Electric Light Power Company, of Valatie, was incorporated by the laws of 1897 to construct reservoirs and create extensive power developments on Kinderhook Creek and its tributaries in Columbia and Rensselaer counties. The surveys for this work are now in process under the direction of L. L. Tribus, chief engineer of the company.

Wallkill River, a branch of Rondout Creek, may be mentioned. This stream rises in New Jersey and flows north, joining Rondout Creek near Rondout. It is proposed to take an additional water supply for Brooklyn Borough of Greater New York from the head waters of this stream in New Jersey. The drainage area south of the New York State line is 210 square miles.

ALLEGHENY RIVER SYSTEM.

The Allegheny River enters the State of New York from Pennsylvania in the southeastern corner of Cattaraugus County and, flowing in nearly a semicircle with its outward curve to the north, passes out of the State in the southwestern part of Cattaraugus County, again entering Pennsylvania. Its principal tributary from the north is Conewango Creek, which receives the outlet of Chautauqua Lake and of Cassadaga Creek as tributaries. Little Valley, Great Valley, and Olean creeks are also tributaries in New York. None of these streams is of especial importance for water power.

Chautauqua Lake, 20 miles in length and from 1 to nearly 2 miles in width, is distant from Lake Erie at its northern extremity only about 9.5 miles. Its elevation above tide water is 1,297 feet, while that of Lake Erie is 573 feet. Hence Chautauqua Lake is 724 feet above Lake Erie. The Conewango Creek, at the south line of the State, has an elevation of 1,243 feet. The fall from Chautauqua Lake to the southern boundary of the State along the drainage line is therefore only 54 feet. The drainage area of the Chautauqua outlet at the foot of Chautauqua Lake is 178 square miles, and of the Chautauqua outlet below Cassadaga Creek 343 square miles.

SUSQUEHANNA RIVER SYSTEM.

The head waters of the North Branch of Susquehanna River lie chiefly in the State of New York, the drainage area in this State being taken at 6,267 square miles. The main stream is considered as rising in Otsego Lake, from which it flows first southwesterly, then westerly with a short portion of its course south of the Pennsylvania line. It finally leaves New York State in Tioga County. The Susquehanna, while one of the large rivers of New York, is not at all important as regards water power. The main river and most of its tributaries in New York flow through a rolling country with fairly uniform declivity. While utilized for small powers in many places, thus far there are no extensive developments on either the main stream or its branches, except at Binghamton, where considerable water power is utilized. The slope of the stream in New York State is shown by following elevations, in feet, above tide water:

Elevations along Susquehanna River in New York State.

	Feet.
At Towanda, a few miles south of the State line	700
At Athens, on Chemung River, near the State line	744
At Otsego Lake	1,193

So far as known, no discharge measurements of this stream have ever been made.

DRAINAGE AREA OF SCHROON RIVER.

one of the important tributaries of the North Branch of Susque-
a River in New York may be mentioned Chenango River, which
in Madison County, and flows south through Madison, Chenango,
Broome counties, emptying into the Susquehanna at Binghamton.
ung River, the chief tributary of the North Branch, is formed
euben County, New York, near Painted Post, by the junction of
Tioga and Cohocton Rivers, whence it pursues a southeasterly
se, joining the Susquehanna near Athens, in Bradford County,
sylvania, just south of the State line. Tioga River rises in Tioga
ty, Pennsylvania, and flows north to join the Cohocton at Painted
Canisteo, the principal New York State tributary of the Tioga,
the main stream 5 miles south of Painted Post. Cohocton River,
h rises in Livingston County, and flows southeast to join the Tioga
ainted Post, is utilized for small powers at a number of places.
area drained by the Cohocton and Canisteo is almost entirely
ded of forest and the streams are in consequence much less valu-
for power than formerly. For a considerable length of time, in
all of 1895, the natural yield of these streams was probably consid-
ly less than one-tenth of a cubic foot per second per square mile.
her tributaries of the Susquehanna in New York State are Owego
Cuyuta creeks, neither of which, although formerly extensively
zed for small powers, is now of great value, largely because of the
edingly slight summer flows.
e following are the drainage areas of the Susquehanna and its
itaries in the State of New York:

Drainage areas of Susquehanna River and its tributaries in New York.

	Square miles.
river below mouth of Chemung River (south of Pennsylvania line)	7,463
area north of Pennsylvania line	6,267
e mouth of Chemung River	4,945
nghamton	2,279
isquehanna	2,024
ineveh	1,789
v mouth of Unadilla River	1,638
v mouth of Oak Creek	212
e mouth of Oak Creek	97
Creek	115
y Valley Creek	121
evas Creek	127
otte River	178
Creek	106
ut Creek	115
illa River	561
rnut Creek	123
ango River at mouth	1,540
ango River above Tioughnioga	685
ango River above Canasawacta Creek	297
ghnioga River at mouth	785

Drainage areas of Susquehanna River and its tributaries in New York—Cont'd.

	Square miles.
Tioughnioga River above mouth of Otselic River	438
Otselic River	259
West Branch of Tioughnioga River	103
East Branch of Tioughnioga River	164
Owego Creek	391
Cayuta Creek	148
Chemung River at junction of Canisteo and Cohocton rivers	1,941
Chemung River at Elmira	2,055
Chemung River at mouth	2,518
Cohocton River at mouth	425
Tioga River at mouth	1,530
Tioga above mouth of Canisteo	750
Canisteo at mouth	780
Tuscarora Creek at mouth	120

DELAWARE RIVER SYSTEM.

The extreme head waters of Delaware River, in New York, are a series of small ponds in Schoharie County, a little north of the village of Stamford. From this point the stream flows southwesterly to Deposit, on the line between Broome and Delaware counties, where it turns south, on which general course it continues until near the Pennsylvania line, whence its course is southwesterly to Port Jervis. This portion of the river is the boundary line between New York and Pennsylvania. At Port Jervis the river passes from New York State, making a sharp turn to the southwest. The declivity of Delaware River is shown by the following elevations above tide water:

Elevations along Delaware River in New York State, showing declivity of the stream.

	Feet.
At Lackawaxen	600
At Deposit	984
At head waters	1,886

The principal tributary in New York State is Pepacton River, which rises in the eastern part of Delaware and Greene counties and flows southwest in a course generally parallel to the main stream. Neversink Creek, the next important tributary in the State, joins the main stream at the State line a mile south of Port Jervis. Neither of these streams has thus far developed any large amount of water power, although there are a number of places where good powers could be developed. The cutting off of the forests of the Delaware drainage area has undoubtedly greatly injured the tributary streams for mill purposes.

The following are the more important drainage areas on the Delaware and its tributaries in New York State:

Drainage areas on Delaware River and tributaries in New York State.

	Square miles.
Total area in New York State	2,580
Main stream below mouth of Neversink River	3,600
Main stream below Port Jervis	3,252
Main stream below junction of East and West branches	1,604
West Branch at mouth	685
West Branch at Deposit, below Oquaga Creek	519
Popacton River at mouth	919
Above mouth of Beaverkill	520
Beaverkill Creek	322
Oquaga Creek	82
Little Delaware Creek	53
Neversink River at mouth	346

By way of concluding the general discussion of the Allegheny, Susquehanna, and Delaware river systems in the State of New York, it may be remarked that these have all been extensively used, either for floating logs or for propelling sawmills, or for both. The clearing up of the drainage areas has, however, long since reduced the lumbering business to nothing. These streams are, therefore, much less extensively utilized than formerly. At present, aside from one or two points, their use is chiefly for propelling small sawmills and gristmills and for other moderate-sized industries. With one or two exceptions, there are no large power developments throughout the whole region.

STREAMS OF LONG ISLAND.

Long Island is chiefly a sandy plain, about 120 miles in length, with a total area of 1,682 square miles. A considerable portion is below an elevation of 100 feet above tide water, although in places it rises to elevations of 300 feet and more. The streams are all small and only a few miles in length, running down from the high land of the middle section to the Atlantic Ocean on the south and to Long Island Sound on the north. As regards water power, the water resources of Long Island have little significance, although there are many places where small powers are utilized for gristmills and other similar uses. The chief value of the inland water of Long Island is for the water supply of the city of Brooklyn.

East River, which connects Long Island Sound with New York Bay, may also be referred to for convenience as a Long Island water resource. The great value of the stream to the commerce of New York is so obvious as to hardly require mention.

The foregoing description of the river systems of New York has

been made as brief as possible, because very complete descriptions have been given in the several monographs relating to New York State which appear in the report on the Water Power of the United States, Tenth Census, 1880. In these reports may be found full details of the several river valleys, with statements as to agricultural production, population, geology, climatology, and many other subjects not touched on here.

AVAILABLE WATER SUPPLY.

RUN-OFF OF NIAGARA RIVER.

The great developments of the Niagara Falls Power Company, authorized by the laws of 1886, have been in part completed, while at the same time the original Niagara Falls power development, now owned by the Niagara Falls Power and Manufacturing Company, has increased greatly in capacity. The laws of 1886, and amendments thereto, have also authorized the taking from Niagara River of large quantities of water for the purpose of creating a water power near the city of Lockport. A ship canal is projected connecting Lakes Erie and Ontario, and the Canadian Government has made a concession for extensive power developments on the Canadian side of the river. Hence it is evident that the future demands for water to be taken from Niagara River and delivered either into the lower river below the Falls or into Lake Ontario independent of the river are very large, and the interest which the people of the State of New York have in the run-off of Niagara River becomes exceedingly important.

The most recent determination of the area of the basin drained by the Great Lakes and of the water surfaces of the lakes themselves is that given in the report of the United States Deep Waterways Commission, from which the following general summary is taken:

Water surface and watershed areas of the basin drained by the Great Lakes.

Lake.	Area of water surface.	Area of watershed.	Total area of basin.
	Square miles.	*Square miles.*	*Square miles.*
Superior	31,800	48,600	80,400
Michigan	22,400	45,700	68,100
Huron	23,200	52,100	75,300
St. Clair	495	6,320	6,815
Erie	10,000	24,480	34,480
Total	87,895	177,200	265,095

That portion of the drainage area of Lake Erie lying within the tate of New York is given as 2,210 square miles. The area of islands ı Niagara River is given as 29 square miles. That portion of the atershed of Niagara River lying within the State of New York has n area of 789 square miles. The area of the river itself, from its ead at Lake Erie to the Falls, is 21 square miles.[1]

The accompanying table gives the precipitation within and in the icinity of the drainage area of the Great Lakes for the years from 392 to 1895, inclusive. In this table a few only of the many precipitaon records which are now available have been used. The records ıere appearing are, it is believed, sufficient to show the mean prepitation of the basin of the Great Lakes for the years indicated. In ıis and subsequent tables the water year[2] is considered as beginning ith the month of December of the preceding year. Thus the water ear of 1879 extends from December, 1878, to November, 1879, inclu-ve. The months from December to May constitute what may be ırmed the storage period. During this time vegetation is inert, the emperature low, with consequent light evaporation. Usually from) to 80 per cent of the precipitation of this period runs off in the ;reams.

June to August, inclusive, is the growing period. Then the temerature is at its highest for the year and vegetation is active. For ıis period only about 20 per cent of the rainfall appears in the ;reams, and some of that is usually the stored ground water of the receding period.

September to November, inclusive, is the replenishing period. The emperature is again relatively low, vegetation inert, and the rainfall ;oes to replenish the stock of ground water, depleted during the growng period, or, after the ground again becomes full, appears as direct :un-off in the streams. In the table the monthly precipitation has been omitted, as it can readily be found in the annual reports of the United States Weather Bureau, and the data have been condensed to show the total quantities for the storage, growing, and replenishng periods, together with the total annual amounts.

---

[1] Report of the United States Deep Waterways Commission, by the commissioners, James B. ngell, John E. Russell, Lyman E. Cooley. Accompanied by the report on technical work and e several topical reports and drawings pertaining thereto. Printed as House Document No. ;, Fifty-fifth Congress, second session, Washington, 1897, pp. 146, 147.

[2] For the drainage area of the Great Lakes in detail reference may be made to an excellent map the basin of the Great Lakes and of St. Lawrence and Hudson rivers in relation to the surnding drainage systems accompanying the report of the Deep Waterways Commission.

[3] Report of the State Engineer and Surveyor of New York, 1895, p. 99.

Precipitation, in inches, within and in the vicinity of the drainage area of the Great Lakes, 1892 to 1895, inclusive.

Locality and year.	December to May.	June to August.	September to November.	Annual
Pokegama Falls, Minn.:				
1892	11.76	6.99	2.86	21.61
1893	9.64	13.06	2.84	25.54
1894	14.72	8.04	9.01	31.77
1895	9.14	12.49	5.70	27.33
Mean				26.56
Duluth, Minn.:				
1892	17.96	11.78	2.39	32.13
1893	11.08	6.86	3.64	21.58
1894	19.44	3.80	8.51	31.75
1895	6.44	9.32	7.70	23.46
Mean				27.23
Minneapolis, Minn.:				
1892	13.78	23.32	2.33	39.43
1893	12.80	9.79	5.68	28.27
1894	15.66	1.73	6.46	23.85
1895	7.72	9.96	5.01	22.69
Mean				28.[illegible]
Green Bay, Wis.:				
1892	14.95	12.47	7.95	35.37
1893	14.85	8.12	7.15	30.12
1894	19.65	8.52	10.46	38.63
1895	10.06	7.52	3.14	20.72
Mean				31.[illegible]
Madison, Wis.:				
1892	18.89	13.34	4.89	37.12
1893	13.37	12.75	5.82	31.94
1894	10.96	6.23	7.61	24.80
1895	5.54	3.88	2.57	11.99
Mean				[illegible]
Milwaukee, Wis.:				
1892	18.17	11.00	5.71	34.88
1893	15.69	10.14	6.06	31.89
1894	15.94	4.81	8.79	29.54
1895	10.34	7.45	5.33	23.12
Mean				[illegible]

n inches, within and in the vicinity of the drainage area of the Great Lakes, 1892 to 1895, inclusive—Continued.

lity and year.	December to May.	June to August.	September to November.	Annual.
	16.03	14.66	5.56	36.25
	13.93	6.85	6.18	26.96
	14.48	3.16	10.30	27.94
	9.58	10.70	7.00	27.28
				26.61
L.:				
	27.26	10.91	6.97	45.14
	24.45	4.52	8.92	37.89
	21.11	4.58	7.82	33.51
	9.08	6.40	8.56	24.04
				35.15
h.:				
	14.52	8.18	7.70	30.38
	20.54	7.18	10.33	38.05
	16.63	2.76	7.21	26.60
	8.92	5.40	4.70	19.02
				28.51
Mich.:				
	16.57	8.93	4.97	30.47
	19.02	6.79	8.68	34.49
	19.84	4.07	11.08	34.99
	10.89	4.88	6.17	21.94
				30.47
h.:				
	16.03	3.53	8.89	28.45
	14.81	9.16	7.41	31.38
	24.65	5.25	9.31	39.21
	16.35	7.04	8.89	32.28
				32.83
a.:				
	12.39	10.30	6.43	29.12
	15.61	7.94	8.31	31.86
	17.80	6.83	9.80	34.43
	11.66	4.59	8.98	25.23
				30.16

Precipitation, in inches, within and in the vicinity of the drainage area Great Lakes, 1892 to 1895, inclusive—Continued.

Locality and year.	December to May.	June to August.	September to November.	An
Traverse City, Mich.:				
1892	17.65	10.87	8.61	
1893	17.88	7.07	11.41	
1894	20.62	5.61	9.72	
1895	16.69	4.53	7.85	
Mean				
Cleveland, Ohio:				
1892	19.84	11.91	5.90	
1893	19.09	5.46	7.45	
1894	15.28	5.55	7.77	
1895	9.29	7.59	7.91	
Mean				
Toledo, Ohio:				
1892	17.77	12.76	6.47	[illegible]
1893	10.17	4.81	6.92	[illegible]
1894	14.93	2.78	5.23	[illegible]
1895	9.23	6.24	7.11	[illegible]
Mean				[illegible]
Buffalo, N. Y.:				
1892	22.62	16.93	8.32	47
1893	20.65	8.00	7.87	36
1894	22.47	5.82	12.50	40
1895	14.17	6.23	8.85	29
Mean				38
Rochester, N. Y.:				
1892	17.75	13.41	5.94	37
1893	18.05	9.36	6.02	33
1894	21.26	7.05	7.14	35
1895	16.16	6.84	7.15	30
Mean				34
Oswego, N. Y.:				
1892	15.22	15.33	6.82	37
1893	14.63	9.83	8.01	32
1894	19.55	6.46	11.13	37
1895	15.06	6.25	9.08	30
Mean				[illegible]

in inches, within and in the vicinity of the drainage area of the Great Lakes, 1892 to 1895, inclusive—Continued.

›cality and year.	December to May.	June to August.	September to November.	Annual.
ınitoba:				
.....	6.65	8.70	3.96	19.31
.....	8.25	10.81	4.35	23.41
.....	8.55	3.80	5.84	18.19
.....	8.18	6.62	2.42	17.22
.....	.....	.....	.....	*19.53*
Ontario:				
.....	8.84	7.35	5.31	21.50
.....	8.48	7.39	6.50	22.37
.....	8.20	5.57	8.30	22.07
.....	8.76	7.86	6.05	22.67
.....	.....	.....	.....	*22.15*
ırio:				
.....	12.21	12.29	6.65	31.15
.....	18.64	9.85	7.86	36.35
.....	19.90	3.07	8.44	31.41
.....	11.93	6.24	7.76	25.93
.....	.....	.....	.....	*31.21*

ecipitation data are of special interest because the year e culmination of a period of exceedingly low water. They or a period of four years the precipitation of this basin ıd in consequence the run-off of the tributary streams been exceedingly small. As illustrating this proposition, t refer to the run-off of the Upper Mississippi,[1] where there ir system controlling a drainage area of 3,265 square miles, d about 1885. The rainfall of the area tributary to these ıs indicated by records kept at Leech Lake, Lake Wini- nd Pokegama Falls from 1885 until the present time is, ge, from 24 to 26 inches per year. The highest recorded ipitation is 31.87 inches, at Pokegama Falls in 1894. The the area tributary to the Upper Mississippi reservoirs is e quite similar to that of the region tributary to Lake Hence the run-off of this reservoir system may be con- representing the run-off of the drainage area of Lake ıd the northern portion of Lakes Michigan and Huron.

ırt of Chief of Engineers U. S. Army for 1896, Part III, p. 1843; also for 1897, Part

The following gives the discharge from these reservoirs for the years 1892 to 1895, inclusive, corresponding with the years of precipitation shown in the table on pages 50 to 53:

Mean rainfall, run-off, and proportion of run-off to rainfall of the area tributary to the Upper Mississippi reservoirs.

Water year.	Mean rainfall on water shed.	Run-off of watershed.	Proportion of run-off to rainfall.
	Inches.	*Inches.*	*Per cent.*
1892	21.33	4.43	20.8
1893	25.42	3.61	14.2
1894	26.63	3.62	13.6
1895	25.11	2.79	11.1
Total	98.49	14.45	.....
Mean	24.62	3.61	14.7

The table shows that during the years 1892 to 1895, inclusive, the mean run-off of the Upper Mississippi watershed was only 3.61 inches on the total watershed. These figures, however, are subject to correction because the state of the reservoirs at the beginning and ending of the four-year period is not given in the report of the United States engineers, from which these data are taken. This correction, however, can not be very large, because the reservoirs are so operated as to be emptied, generally speaking, each year. In considering the run-off of these Upper Mississippi reservoirs, due consideration should be given to the fact that the water area of the reservoirs is 585 square miles, or nearly 18 per cent of the whole. For Lakes Superior, Michigan, Huron, St. Clair, and Erie we have a total water surface of 87,895 square miles, with a total drainage area, including the surface of the lakes, of 265,095 square miles. The water surface of these several lakes is, therefore, about 33 per cent of the entire area of the basin, or nearly double the relative area of water surface and drainage area for the Upper Mississippi reservoirs. With other conditions the same, this fact would probably lead to a somewhat greater proportion of run-off from the Great Lakes.

By way of further illustrating the yield of streams in the vicinity of the Great Lakes drainage area, we may refer to the run-off of the Des Plaines River as given in the table on page 64. This stream has been measured by the Chicago drainage commission, with certain intermissions, as shown by the table, since January, 1886, the drainage area above the point of measurement being 633 square miles. The drainage area comprises a long and narrow, flat region extending from near Chicago to a few miles north of Milwaukee, the eastern line being for the entire distance nearly parallel to Lake Michigan and in

places only 2 or 3 miles distant therefrom. The area drained by the Des Plaines River is large enough to give a fair idea of the average yield of streams tributary to Lake Michigan in northern Illinois and Indiana, western Michigan, and southern and central Wisconsin. In 1893, with a mean rainfall on the drainage area of 39.96 inches, the total run-off was 10.14 inches, of which 8.61 inches occurred during the storage period from December to May, inclusive. In 1894, with a total rainfall of 27.94 inches, the total run-off was 7.70 inches, of which 7.54 inches occurred in the storage period. For the year 1895 the total rainfall was 27.28 inches. The run-off data of this year are unfortunately incomplete, but taking into account the sequence of the rainfall it is clear that the total run-off for that year did not exceed about 2.0 to 2.5 inches. The effect of the three dry years 1893, 1894, and 1895 in the Des Plaines drainage area is shown by the record of 1896, where, with a total rainfall of 39.58 inches, the total run-off was only 6.69 inches, of which 5.39 inches occurred in the storage period. These figures indicate that the ground water of the Des Plaines area must have been so low at the end of 1895 as to absorb a large portion of the heavier rainfall of 1896 before any great amount could appear as run-off.[1]

Rainfall, run-off, evaporation, and mean temperature of Muskingum River, as measured by the United States engineers, from 1888 to 1895, inclusive.

[In inches on the watershed.]

Month.	1888.				1889.				1890.			
	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.
				°				°				°
December	1.94	0.18	.....	30.8	1.50	0.84	.....	31.5	3.01	1.48	.....	41.1
January	3.96	1.24	.....	23.5	3.63	1.89	.....	31.7	4.52	3.53	.....	37.0
February	1.91	1.12	.....	29.4	1.55	1.42	.....	24.0	5.84	3.73	.....	37.5
March	4.05	1.38	.....	32.5	1.71	0.71	.....	39.0	4.38	4.23	.....	32.0
April	1.75	0.80	.....	46.9	2.23	0.88	.....	47.9	3.41	2.00	.....	48.8
May	3.55	0.45	.....	58.4	2.90	0.28	.....	59.1	6.61	3.10	.....	56.9
	17.16	*5.17*	*11.99*	*36.9*	*13.52*	*6.02*	*7.50*	*38.9*	*27.77*	*18.07*	*9.70*	*42.2*
June	2.66	0.29	.....	68.4	4.79	0.47	.....	65.7	5.27	1.64	.....	70.7
July	5.81	0.81	.....	70.4	5.35	0.55	.....	71.7	3.06	0.51	.....	70.9
August	5.84	0.67	.....	68.8	1.98	0.22	.....	67.0	5.35	0.49	.....	66.6
	14.31	*1.77*	*12.54*	*69.2*	*12.12*	*1.24*	*10.88*	*68.1*	*13.68*	*2.64*	*11.04*	*69.4*
September	3.28	0.61	.....	58.0	4.17	0.14	.....	61.3	6.86	2.28	.....	60.6
October	3.25	0.77	.....	45.6	2.35	0.14	.....	46.5	6.20	2.01	.....	51.1
November	4.61	2.01	.....	41.1	3.72	0.68	.....	40.2	2.46	1.84	.....	42.0
	11.14	*3.39*	*7.75*	*48.2*	*10.24*	*0.96*	*9.28*	*49.3*	*15.52*	*6.13*	*9.39*	*51.2*
Year	42.61	10.33	32.28	47.8	35.88	8.22	27.66	48.8	56.97	26.84	30.13	51.3

[1] For details of the measurements of the Des Plaines River see data pertaining to rainfall and stream flow, by Thomas T. Johnston, Journal Western Soc. C. E., Vol. I (June, 1896).

Rainfall, run-off, evaporation, and mean temperature of Muskingum River, as measured by the United States engineers, from 1888 to 1895, inclusive—Cont'd.

Month.	1891.				1892.				1893.			
	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.
				°				°				°
December	2.45	0.91	.....	29.3	2.62	1.03	.....	37.8	1.62	0.24	.....	28.1
January	2.52	2.40	.....	31.2	2.40	0.74	.....	22.8	3.01	0.37	.....	18.0
February	4.43	4.56	.....	34.8	2.76	2.40	.....	33.0	6.38	4.70	.....	28.1
March	3.19	2.52	.....	33.5	3.10	1.26	.....	32.7	2.31	2.55	.....	30.9
April	1.74	1.58	.....	49.8	2.24	1.38	.....	46.0	5.94	2.22	.....	40.4
May	2.39	0.45	.....	55.8	7.27	2.25	.....	57.9	5.78	4.05	.....	57.0
	16.72	*12.42*	*4.30*	*39.1*	*20.39*	*9.06*	*11.33*	*38.4*	*25.04*	*14.13*	*10.91*	33.3
June	7.56	0.91	.....	69.0	7.05	2.30	.....	71.6	3.08	0.78	.....	70.2
July	3.92	0.51	.....	67.6	5.44	0.75	.....	70.9	2.92	0.28	.....	72.4
August	2.08	0.35	.....	68.7	4.05	0.60	.....	70.0	2.31	0.16	.....	68.5
	13.56	*1.77*	*11.79*	*68.4*	*16.54*	*3.65*	*12.89*	*70.8*	*8.31*	*1.22*	*7.09*	71.1
September	1.08	0.17	.....	66.5	2.33	0.28	.....	62.2	1.56	0.14	.....	63.4
October	1.25	0.26	.....	49.7	0.80	0.20	.....	50.1	5.51	0.46	.....	52.4
November	4.75	0.94	.....	38.6	1.68	0.19	.....	37.8	1.94	0.25	.....	38.4
	7.08	*1.37*	*5.71*	*51.6*	*4.81*	*0.67*	*4.14*	*50.0*	*9.01*	*0.85*	*8.16*	51.1
Year	37.36	15.56	21.80	49.5	41.74	13.38	28.36	49.4	42.36	16.20	26.16	48.1

Month.	1894.				1895.				Mean.			
	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.	Rainfall.	Run-off.	Evaporation.	Temperature.
				°				°				°
December	2.59	1.54	.....	31.7	3.01	0.36	.....	33.2	.....	.....	.....	—
January	2.11	1.00	.....	33.2	3.99	1.67	.....	22.4	.....	.....	.....	—
February	2.95	2.19	.....	27.6	0.89	0.12	.....	19.1	.....	.....	.....	—
March	2.20	1.41	.....	43.9	1.87	1.20	.....	33.2	.....	.....	.....	—
April	2.45	0.75	.....	49.8	1.76	0.59	.....	50.6	.....	.....	.....	—
May	4.63	0.74	.....	58.3	1.52	0.10	.....	60.5	.....	.....	.....	—
	16.94	*7.63*	*9.30*	*40.7*	*13.04*	*4.04*	*9.00*	*36.5*	*18.82*	*9.57*	*9.25*	38.1
June	2.12	0.42	.....	70.6	2.87	0.17	.....	71.6	.....	.....	.....	—
July	1.77	0.13	.....	73.1	2.52	0.21	.....	69.6	.....	.....	.....	—
August	0.67	0.11	.....	70.1	3.75	0.11	.....	71.7	.....	.....	.....	—
	4.56	*0.66*	*3.90*	*71.3*	*9.14*	*0.49*	*8.65*	*71.0*	*11.53*	*1.68*	*9.85*	[illegible]
September	4.12	0.15	.....	67.2	2.39	0.13	.....	67.2	.....	.....	.....	—
October	2.19	0.10	.....	54.3	1.38	0.08	.....	45.3	.....	.....	.....	—
November	2.71	0.16	.....	36.8	3.89	0.16	.....	40.8	.....	.....	.....	—
	9.02	*0.41*	*8.61*	*52.4*	*7.66*	*0.37*	*7.29*	*51.1*	*9.31*	*1.77*	[illegible]	[illegible]
Year	30.51	8.70	21.81	51.3	29.84	4.90	24.94	48.8	30.66	13.42	26.64	[illegible]

. the table is given the rainfall and run-off record for Muskingum
er, in Ohio, as measured at Zanesville[1] for the years 1888 to 1895,
usive, the area of the watershed above the point of measurement
ig 5,828 square miles. The head waters of the stream are not far
n Lake Erie and on the dividing line between the hill country of
east and the prairie country of the Mississippi Valley. Hence
stream represents conditions applicable to the run-off of the Ohio
ams tributary to Lake Erie. The rainfall record as used in this
e is the mean of the records kept at Akron, Canton, Newcomers-
n, and Wooster, and may be considered to represent fairly well
mean precipitation of the Muskingum drainage area. For the
r 1892 the total rainfall was 41.74 inches and the total run-off 13.38
nes, of which 9.06 occurred in the storage period. In 1893 the total
ifall was 42.36 inches, with a total run-off of 16.20 inches, the run-
of the storage period being 14.13 inches. In 1894 the rainfall
pped to a total of 30.51 inches and the run-off to a total of 8.70
nes, of which 7.63 inches occurred in the storage period. In 1895
total rainfall was 29.84 inches and the total run-off 4.90 inches, of
ch 4.04 inches occurred during the storage period.

Genesee River, while not tributary to the Great Lakes above Niagara
er, may still be cited as showing that at times the run-off of
ams tributary to the Great Lakes is quite low. Referring to the
le on page 70, giving the rainfall and run-off of Oatka Creek, a
utary of the Genesee, we learn that in the water year 1891, with
infall of 38.12 inches, the run-off was 14.05 inches. In 1892, with
infall of 41.69 inches, the run-off was 15.42 inches.

aking the record of Genesee River proper, as given in the table
page 58, we learn that for the water year 1894, with a mean pre-
tation above the point of measurement of 47.79 inches, the run-off
19.38 inches, of which 15.73 inches occurred in the storage period.
895 the rainfall dropped to 31 inches and the total run-off to 6.67
nes. In 1880 Hemlock Lake, a tributary of Genesee River, with a
nage area of 43 square miles and a total rainfall of 21.99 inches,
e a run-off of only about 3.4 inches.

---

rvey of the Miami and Erie Canal, the Ohio Canal, etc. Report of Capt. Hiram M. Chitten-
Corps of Engineers, U. S. Army, January 20, 1896, printed as House Document No. 278,
-fourth Congress, first session, p. 42.

Rainfall, run-off, evaporation, and mean temperature of Genesee River from December, 1893, to November, 1896, inclusive.

[In inches on the watershed.]

Month.	1894.				1895.				1896.			
	Rainfall.	Run-off.	Evaporation.	Mean temperature	Rainfall.	Run-off.	Evaporation.	Mean temperature.	Rainfall.	Run-off.	Evaporation.	Mean temperature
				°				°				°
December	3.39	2.34	.....	26.8	2.47	0.61	.....	29.0	3.80	1.32	.....	[illegible]
January	3.91	1.40	.....	28.4	3.26	0.66	.....	1.91	2.29	0.47	.....	[illegible]
February	2.93	0.86	.....	20.1	1.22	0.22	.....	14.5	3.56	0.01	.....	[illegible]
March	1.62	3.31	.....	37.9	1.72	1.94	.....	28.0	4.00	3.00	.....	[illegible]
April	7.22	3.39	.....	42.1	2.10	2.01	.....	44 2	1.62	3.38	.....	[illegible]
May	8.64	4.43	.....	54.5	2.43	0.19	.....	57.8	2.57	0.17	.....	[illegible]
	27.71	*15.73*	*11.98*	*35.0*	*13.20*	*5.63*	*7.57*	*32.1*	*17.84*	*9.25*	*8.59*	[illegible]
June	2.51	1.10	.....	64.6	4.57	0.13	.....	67.5	3.52	0.39	.....	[illegible]
July	3.70	0.14	.....	67.9	2.57	0.11	.....	64.1	4.90	0.24	.....	[illegible]
August	1.74	0.23	.....	63.4	3.99	0.12	.....	66.5	1.86	0.20	.....	[illegible]
	7.95	*1.46*	*6.49*	*65.3*	*11.13*	*0.36*	*10.77*	*66.0*	*10.28*	*0.83*	*9.45*	[illegible]
September	0.97	0.93	.....	61.6	1.96	0.10	.....	61.7	5.22	0.16	.....	[illegible]
October	3.50	0.44	.....	49.5	1.30	0.11	.....	41.8	4.08	1.74	.....	[illegible]
November	1.66	0.82	.....	32.1	3.41	0.47	.....	38.0	3.26	0.82	.....	[illegible]
	12.13	*2.19*	*9.94*	*47.7*	*6.67*	*0.68*	*5.99*	*47.2*	*12.56*	*2.72*	*9.84*	[illegible]
Year	47.79	19.38	28.41	45.7	31.00	6.67	24.33	44.3	40.68	12.80	27.88	[illegible]

These figures are cited to show that in years of low rainfall the run-off of streams tributary to the Great Lakes is very low, and as a consequence the run-off of Niagara River will probably be affected thereby. At present the data are insufficient for showing what the run-off of Niagara River really is.

The most elaborate measurements thus far made are those of the Lake Survey in 1867 and 1868, which are, however, extremely unsatisfactory. According to these measurements the mean discharge, rainfall, and evaporation from the Great Lakes for the year 1868, in cubic feet per second, were as follows:[1]

Mean discharge, rainfall, and evaporation from the Great Lakes for the year 1868, in cubic feet per second.

Lakes.	Mean discharge.	Total rainfall on basin.	Evaporation from surface.
Superior	86,000	171,430	27,690
Huron and Michigan	225,000	251,450	59,890
Erie	265,000	100,540	14,310
Total	.....	523,220	101,890

[1] These figures are derived from Mr. Cooley's Lakes and Gulf Waterways, as corrected and given in the Journal of the Assoc. of Eng. Soc., Vol. VIII (March, 1889), p. 132.

According to the Deep Waterways Commission's tabulations of available records of heights of the Great Lakes, it appears that the water levels fluctuate through a series of years to the extent of about 4.5 feet. For the present discussion we are chiefly concerned with the fluctuations of Lake Erie, which control the discharge of Niagara River. By examination of the records of mean monthly elevation of Lake Erie at Buffalo from 1887 to 1897, as kept by the United States engineer's office at Buffalo, it appears that the highest mean monthly elevation during these years was for June, 1887, when the mean lake surface was +0.92. The lowest mean monthly elevation for the period was for March, 1896, when the mean for the month was −2.36. The range in the mean monthly elevations for this period was 3.28 feet.

Mean monthly elevation of Lake Erie at Buffalo.

[In feet with reference to datum.]

Month.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.	1897.
December	.....	−0.26	−0.31	−0.71	−0.31	−1.35	−1.01	−0.92	−1.23	−2.08	−1.82
January	.....	−0.48	−0.49	−0.13	−0.57	−1.44	−1.78	−0.92	−1.36	1.93	−1.47
February	.....	−1.09	−0.71	−0.31	−0.69	−2.13	−1.83	−1.30	−2.05	−2.00	−1.90
March	+0.60	−1.02	−1.10	−0.07	−0.55	−1.93	−1.52	−1.19	−2.13	2.36	−1.30
April	+0.56	−0.36	−0.72	+0.18	−0.43	−1.09	−0.86	−1.00	−1.92	1.83	−0.87
May	+0.78	−0.21	−0.56	+0.54	−0.65	−0.65	−0.14	−0.50	−1.57	−1.38	−0.45
	+0.64	*−0.57*	*−0.65*	*−0.08*	*−0.53*	*−1.43*	*−1.19*	*−0.97*	*−1.71*	*1.93*	*1.32*
June	+0.92	−0.04	−0.09	+0.87	−0.67	+0.16	+0.21	−0.11	1.47	1.37	0.46
July	+0.76	+0.18	−0.05	+0.59	−0.49	+0.37	+0.08	−0.24	−1.49	−1.19	−0.48
August	+0.36	+0.07	−0.14	+0.12	−0.78	+0.00	−0.51	−0.71	−1.63	0.96	−0.96
	+0.68	*+0.07*	*−0.09*	*+0.53*	*−0.65*	*+0.18*	*−0.07*	*−0.36*	*−1.53*	*−1.18*	*0.63*
September	+0.05	−0.27	−0.52	−0.23	−0.95	−0.27	−0.76	−0.08	1.61	−1.38	1.40
October	+0.11	−0.46	−1.03	−0.25	−1.32	−0.00	−0.87	−0.88	−1.85	−1.64	1.36
November	−0.44	−0.61	−1.03	−0.05	−1.38	−0.98	−0.88	−1.06	−2.34	−1.61	−1.24
	−0.09	*−0.45*	*−0.86*	*−0.17*	*−1.22*	*−0.62*	*−0.84*	*−0.67*	*−1.93*	*−1.54*	*−1.33*
Year	*a*+0.41	−0.38	−0.56	+0.05	−0.73	−0.83	−0.82	−0.74	−1.72	−1.64	−1.15

a Mean of nine months.

Temporarily much greater fluctuations have been experienced, due largely to wind action, to which Lake Erie, on account of its shallowness, and the fact that its general direction is favorable for the sweep of the prevailing winds, is peculiarly subject. In regard to the measurements of the Lake Survey, it may be remarked that they indicate large variations in discharge from all of the lakes, from the effects of winds and other disturbing causes, but give little clew to the quantities at either of the extremes of high or low water. According to Lyman E. Cooley the extreme low-water discharge is probably 20 to 30 per cent less than the Lake Survey figures, and extreme high water 20 to 30 per cent more.

Measurements of the amount of water flowing in Niagara River were begun in December, 1891, at a time when the water in Lake Erie was very low and the conditions were considered especially favorable for

minimum discharge. The results are given in the Annual Report of the Chief of Engineers, United States Army, for 1893, Part VI, pp. 4364–4371. The point selected was about 1,000 feet below the International Bridge at Black Rock, near the foot of Squaw Island, at which point the river is free from eddies. Niagara River, on leaving Lake Erie, has a nearly straight channel about 2,000 feet wide for the first 2 miles. The fall in this section is from 4 to 5 feet, and the velocity ranges from 7 miles per hour at the upper end to about 5 miles at the lower end. The point was chosen, after careful consideration, as the point in that vicinity least subject to disturbance. In taking the cross sections, the width, which varies slightly with different stages of the river, was actually determined for gage readings 1 foot apart, and for extreme points the width was determined by interpolating values derived from the known slope of the river banks. A local gage was established at the draw pier of the International Bridge by setting gage boards on each side of the pier, with the zeros of the gages on the same level. The local gage was read at the beginning and close of all velocity observations, and the gage at Buffalo was read at 7 a. m. and 1 and 7 p. m. The zero of this latter gage is at the mean level of Lake Erie, or 572.23 feet above mean tide at New York, in the Erie Canal levels, or as used by the Government engineers, 572.96 feet. During the velocity observations in December, 1891, Lake Erie was about 1.5 feet below its mean level, and is stated not to have been seriously affected by strong winds. Still the daily record shows that there must have been some wind action. The current velocities were obtained after the methods used by the Mississippi River Commission and described in their reports, all velocity observations being taken with a current meter, with electrical appliances for recording the number of revolutions. The following are some of the results obtained:[1]

Mean heights and discharge of Niagara River.

Date.	Mean height on local gage.	Mean height on Buffalo gage.	Discharge per second.
1891.	*Feet.*	*Feet.*	*Cu. feet.*
December 24	0.05	—2.95	164,648
December 14	0.65	—1.85	191,822
December 21	0.735	—1.75	193,522
December 20	0.835	—1.75	201,433
December 22	1.125	—1.45	208,597
December 10	1.33	—0.50	218,353
1892.			
May 19	1.562	—0.80	213,180
May 7	1.750	—0.85	218,988
May 24	2.292	+0.15	236,762

[1] Annual Report of Chief of Engineers, United States Army, 1893, Part VI, p. 4365.

The table shows (1) a variation in lake elevation, as indicated in Buffalo gage, from —2.95 on December 24, 1891, to +0.15 on May 1892, a range of 3.10 feet; (2) a variation in discharge of 72,114 bic feet per second. There are some discrepancies in the results ich it is not necessary to discuss at length; but in the absence of re satisfactory data we may assume, in view of the foregoing idence as to the small run-off of streams tributary to and in the inity of the Great Lakes, that the figures obtained in the fall of 91 and spring of 1892 are probably more nearly correct than the ger figures of the Lake Survey. By plotting the observed disarges a mean discharge curve has been obtained, from which the scharge of the river at points within the range of the observation n be taken off, when one has the tabulated heights of the Buffalo ge before him. At present these measurements are, on the whole, t considered sufficiently exact to justify the labor of preparing abulation of this character.[1]

Referring to the table on page 58, it is learned that the rainfall that portion of the basin of the Great Lakes tributary to Niagara ver was, for 1868, 523,220 cubic feet per second, and the evaporan from the water surface of the lakes tributary to Niagara River s 101,890 cubic feet per second. Hence the evaporation from the e surfaces was nearly 20 per cent of the rainfall on the whole basin. suming for the moment the truth of these figures, we have 80 per t of the total rainfall from which the land evaporation must be ucted before anything can run off. Again assuming the land poration at 1.70 feet, there results a loss from this source alone of ,000 cubic feet per second; adding to this the evaporation loss from water surfaces gives a total evaporation loss of 399,890 cubic feet second. The run-off is the difference between rainfall and total poration losses. If, therefore, the land evaporation was 1.7 feet the year 1868, the run-off would have been in reality only 123,330 bic feet per second instead of 265,000 cubic feet per second, as

---

here have been a number of independent measurements of volume of the Niagara, and gh the results differ widely, they probably do not differ more than the actual volume of the r at various stages of Lake Erie.

ell (1841 ?) quotes Ruggles as authority for a volume of 250,000 cubic feet per second.

R. Blackwell, computed by Allen (Am. Jour Sci., 1841), obtains 374,000 cubic feet per second. work was afterwards recomputed by D. F. Henry, who obtained 244,797 cubic feet per nd.

the annual report of the Chief of Engineers, United States Army, for 1867-68, D. F. Henry s as a result of observations in August and September, 1867, 242,494 cubic feet per second. ar later he recomputed from the same data, and obtained 240,192 cubic feet per second. He made a new measurement by a different method (see Report for 1868-69) from which he ined two results, 304,307 and 258,586 cubic feet per second.

F. Reynolds (annual report of the Chief of Engineers, United States Army, 1870?), gives the lt of observations from June to September, 1869, 212,860 cubic feet per second.

the annual report of the Chief of Engineers, United States Army, for 1871, there is mention result, without date of measurement, 245,296 cubic feet per second.

the annual report of the Chief of Engineers, United States Army, for 1891-92, Quintus, as a t of gaging, gives the volume, reduced to mean stage, as 232,800 cubic feet per second.

Casimir S. *Gzowski, from continuous* observations at the International Bridge, 1870-1878, *an average discharge for that* period of 246,000 cubic feet per second.

determined by the Lake Survey. These figures, while not conclusi are suggestive, so much so, indeed, that taking into account all th conditions it seems clear that in a series of years of minimum rainfa the run-off of the Great Lakes, tributary to Niagara River, may be as low as from 6 to 9 inches a year on the watershed. At the form figure the mean discharge would be about 177,700 cubic feet pe second.[1]

As an additional source of loss from the Great Lakes the propos ultimate diversion of 10,000 cubic feet per second through the Chica drainage canal to the head waters of Illinois River may be refer to. Thus far the discussion of such loss has been mainly conducte on the supposition that the mean discharge of the Great Lakes at Niagara was about 265,000 cubic feet per second. If this were true the ultimate injurious effect of such diversion could only appear during a series of extremely dry years. The author can not but think that this whole question of the run-off of Niagara River has become fogged by a discussion based thus far purely on averages. What we really want to know is the run-off of a cycle of dry years. With such data we can compute the effect of a given diversion more satisfactorily than when dealing with means.

With a cycle of rainfall years, either high or at about the average, it is probable that very little effect from such diversion will be observed, the consensus of opinion at the present time apparently being that it will not exceed about 0.3 to 0.4 foot in depth over the areas affected. Owing to the balancing of conditions due to the immense pondage of the Great Lakes, and which requires years in order to complete a cycle, it is uncertain whether the abstraction of 10,000 cubic feet per second at Chicago would be especially detrimental at Niagara Falls although in years of extreme low flow it is probable that it would be easily apparent. If, however, the minimum flow of Niagara River is really as low as 150,000 to 180,000 cubic feet per second, it is clear that the loss of 10,000 cubic feet per second will be a matter worth taking into account.

In the discussion of the effect of diverting 10,000 cubic feet per second at Chicago on the levels of the Great Lakes, by Lyman E. Cooley, which appears in the proceedings of the annual convention of the International Deep Waterways Association, held at Cleveland in September, 1895, it is stated that assuming the correctness of the

[1] By way of illustrating further the probable inaccuracy of the Lake Survey figures, it may be pointed out that if the determination of evaporation from the water surfaces at 101,800 cubic feet per second and run-off at 265,000 cubic feet per second for the year 1868 is correct, the total outgo from these two sources was 368,800 cubic feet per second, leaving the land evaporation for that year at 156,330 cubic feet per second, or at 0.9 foot over the watershed.

By studying the evaporation of the Upper Mississippi reservoirs, the Des Plaines and Muskingum rivers, and other streams herein referred to, it will readily be seen that it is exceedingly improbable that a land evaporation as low as 0.9 foot ever occurred over the whole watershed of the Great Lakes.

ures derived from the Lake Survey placing the mean discharge of Clair River at 225,000 cubic feet per second, the abstraction of ,000 cubic feet per second would diminish the mean outflow in St. air River by nearly 4.5 per cent and in Niagara River by about '5 per cent. Mr. Cooley says that, reasoning on lines obvious to ose unacquainted with hydraulic principles, it is apparent that the ling depth in the rivers at mean level can not be lessened by an iount greater than the percentages just stated; but if we consider e question as an hydraulic proposition, taking into account the lation of mean radius to area and perimeter, it is apparent that the 'ect on lake levels would be only a fraction of that indicated by the duction in volume.

The literature of the discharge of Niagara River and of the probable 'ect on the lake levels of abstracting 10,000 cubic feet per second at iicago has now grown so extensive as to preclude further discussion the question here. Those wishing to pursue the subject further ay consult the references given in the footnote. Concluding the ibject, it may be stated that the studies of the Lake Survey indicate mean discharge of Niagara River of about 265,000 cubic feet per cond, with a range above and below the mean of from 20 to 30 per ınt. The only measurements since made were those of December ı May, 1891–92, which indicate a minimum discharge as low as or ren lower than 141,000 cubic feet per second, this latter figure greeing fairly well with theoretical considerations derived from resent knowledge of the actual minimum run-offs of contiguous rainage areas.[1]

[1] For literature of discharge of Great Lakes and allied questions see (1) Repts. Chief of Engnrs. 1868, 1869, 1870, and 1882; (2) Repts. Chief of Engnrs., 1893; (3) Eng. News. Vol. XXIX (March 2, 1893); (4) The Lakes and Gulf Waterways, by L. E. Cooley; (5) The level of the Lakes as affected by the proposed Lakes and Gulf waterway, a discussion before the Western Society of Engineers, in Jour. of the Assn. of Eng. Socs., Vol. VIII (Mch., 1889); (6) An enlarged waterway between the Great Lakes and the Atlantic seaboard, by E. L. Corthell, with discussion, in Jour. of the Assn. of Eng. Socs., Vols. X and XI (April, June, and December, 1891, and July, 1892); (7) Lake level effects on account of the sanitary canal at Chicago, by L. E. Cooley, in Proc. Internat. Deep Waterways Con., at Cleveland, Sept., 1895; (8) A technical brief, by Thomas T. Johnston, covered by the preceding reference; (9) Papers by William Pierson Hudson, on an Enlarged waterway between the Great Lakes and the Atlantic seaboard, pamphlets, 1890 and 1893.

Rainfall and run-off of Des Plaines River, as determined by the Chicago drai commission, from 1886 to 1897.

[Inches on watershed.]

Months.	1886.		1887.		1888.		1889.		1890.		18	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rain-fall.	R
December	.....	.....	1.76	0.00	3.67	.....	1.94	0.00	1.90	.....	.....	
January	.....	.....	3.13	0.89	1.56	1.37	1.64	0.29	.....	.....	.....	
February	.....	.....	5.10	5.59	1.51	2.50	1.31	0.01	.....	.....	.....	
March	.....	.....	0.89	2.64	2.99	4.84	1.43	0.43	.....	.....	.....	
April	.....	.....	0.46	0.52	.....	.....	2.35	1.13	.....	.....	.....	
May	.....	.....	1.38	0.13	.....	.....	5.38	0.39	.....	.....	.....	
			12.72	*9.77*	.....	.....	*14.05*	*2.25*	.....	.....	.....	
June	0.94	0.16	1.63	0.02	.....	.....	2.93	1.26	.....	.....	.....	
July	1.53	0.14	1.05	0.33	.....	.....	9.56	1.09	2.57	.....	.....	
August	3.38	0.01	3.35	0.18	.....	.....	0.39	0.45	2.58	0.02	.....	
	5.85	*0.31*	*6.03*	*0.53*	.....	.....	*12.88*	*2.80*	.....	.....	.....	
September	6.93	0.03	4.03	0.22	0.98	.....	2.75	0.00	1.39	0.01	.....	
October	1.42	0.01	2.03	0.63	2.95	0.00	1.82	0.00	4.20	0.02	.....	
November	1.66	0.00	2.41	.....	2.89	0.00	3.49	0.01	1.59	0.04	.....	
	10.01	*0.04*	*8.47*	.....	*6.82*	.....	*8.06*	*0.01*	*7.18*	*0.07*	.....	
Year	.....	.....	27.22	.....	.....	.....	34.99	5.06	.....	.....	.....	

Months.	1892.		1893.		1894.		1895.		1896.		18
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rain. fall.	Run-off.	Rain-fall.
December	.....	.....	1.63	0.04	2.14	0.27	1.66	.....	6.76	1.80	0.16
January	.....	.....	2.08	0.01	1.55	0.50	2.15	.....	1.12	0.26	4.53
February	.....	.....	2.44	0.31	2.13	1.06	1.60	0.21	3.48	1.06	2.22
March	.....	.....	1.69	5.15	2.66	3.05	1.32	0.71	1.26	1.11	3.56
April	.....	.....	4.16	1.79	2.65	0.76	0.86	0.29	2.79	0.77	2.22
May	6.77	4.24	1.93	1.31	3.35	1.90	1.99	0.11	4.16	0.39	0.84
			13.93	*8.61*	*14.48*	*7.54*	*9.58*	.....	*19.57*	*5.39*	*13.51*
June	10.58	6.04	3.59	1.37	1.96	0.08	1.79	0.00	2.82	0.09	.....
July	2.23	0.79	3.08	0.14	0.60	0.01	2.42	0.00	3.61	0.02	.....
August	1.85	0.03	0.18	0.00	0.60	0.00	6.49	0.01	3.52	0.06	.....
	14.66	*6.86*	*6.85*	*1.51*	*3.16*	*0.09*	*10.70*	*0.01*	*9.95*	*0.17*	.....
September	1.34	0.02	1.98	0.00	8.28	0.06	0.89	0.06	6.54	0.32	.....
October	1.54	0.00	1.75	0.02	0.84	0.00	0.51	0.00	1.36	0.33	.....
November	2.68	0.02	2.45	0.00	1.18	0.01	5.60	0.00	2.16	0.48	.....
	5.56	*0.04*	*6.18*	*0.02*	*10.30*	*0.07*	*7.00*	*0.06*	*10.06*	*1.13*	.....
Year	.....	.....	26.96	10.14	27.94	7.70	27.28	.....	39.58	6.69	.....

Evaporation from the Des Plaines watershed, as given by differences between rainfall and run-off in the preceding table.

Inches in watershed.

Water year	December to May	June to August	September to November	Total
1886		[illegible]	[illegible]	
1887	[illegible]	[illegible]		
1889	[illegible]	[illegible]	[illegible]	[illegible]
1890			[illegible]	
1892		[illegible]	[illegible]	
1893	[illegible]	[illegible]	[illegible]	[illegible]
1894	[illegible]	[illegible]	[illegible]	[illegible]
1895		[illegible]	[illegible]	
1896	[illegible]	[illegible]	[illegible]	[illegible]

RUN-OFF OF ST. LAWRENCE RIVER.

According to the report of the Deep Waterways Commission, the area of the water surface of Lake Ontario is 7,450 square miles, and the area of the tributary watershed, exclusive of the area of the lake itself, 25,530 square miles. The total area of the drainage basin, including both land and water surfaces, is 32,980 square miles. The area of the water surface of St. Lawrence River from Gallops Rapids to Montreal[1] is given at 220 square miles, and the area of the tributary watershed at 5,710 square miles; hence the total area of the basin of the St. Lawrence from Gallops to Montreal becomes 5,930 square miles.

In the foregoing figures Lake Ontario is considered as beginning in Niagara River, at the foot of Niagara Falls and terminating at the head of Gallops Rapids, whence the following subdivisions of water-surface area are derived: Niagara River, 5 square miles; Lake Ontario proper, 7,260 square miles; St. Lawrence River, 185 square miles; giving a total, as above, of 7,450 square miles.

Of the total area of watershed of 25,530 square miles, 14,275 square miles lie within the State of New York and 11,255 square miles in the Province of Ontario. The standard low-water elevation of Lake Ontario is taken as 244.53 feet, and the standard high-water elevation as 249.04 feet above tide.

St. Lawrence River is considered as beginning at Gallops Rapids. The following table gives the elevation of water surface at a number of points.[2]

[1] Report of U. S. Deep Waterways Commission, 1897, House Document No. 192, Fifty-fourth Congress, second session, pp. 151-153.

[2] Report of U. S. Deep Waterways Commission, 1897, p. 152.

Elevation above tide of low-water and high-water surface of St. Lawrence River.

Locality.	Standard low water.	Standard high water.
	Feet.	*Feet.*
Ogdensburg	244.28	248.57
Lake St. Francis, at Valleyfield	153.50	155.94
Lake St. Louis, at Melicheville	70.0	77.50
Montreal	23.10	35.78

The area of water surface of the St. Lawrence from Gallops Rapids to Montreal is 220 square miles, and the total area of watershed not included in the surface of the river is 5,710 square miles, of which 3,800 square miles lie in New York, 620 in Ontario, and 1,290 in Quebec. The total area of the drainage basin, including water surface of the river, is 5,930 square miles.

The only measurements as to the discharge of St. Lawrence River thus far made are those of the Lake Survey, which give a mean discharge of 300,000 cubic feet per second. The recent data would indicate that this figure is somewhat too large, as in the Lake Survey discharge of Niagara River. The streams tributary to Lake Ontario, however, issue from a region of heavier rainfall than those tributary to the Upper Great Lakes and, as shown by the run-off tables of this report, are generally much better water yielders. Taking everything into account, it is probable that the minimum discharge of St. Lawrence River will not be less than from 8 to 10 inches over the entire watershed per year. A run-off of 12 inches per year over the entire drainage basin would give a mean discharge of 234,300 cubic feet per second, or a discharge of 0.884 cubic foot per square mile per second. A mean discharge of 300,000 cubic feet per second, as measured by the Lake Survey, would give 1.13 cubic feet per square mile per second. These figures are for the minimum discharge; for years, or cycles of years, of average rainfall the run-off would be more.

RUN-OFF OF INLAND STREAMS OF NEW YORK.

The data for determining the run-off of the inland streams of New York are included in the tables given on pages 67 to 85 and others following. The results of measurements on Oatka Creek, a tributary of Genesee River, with a drainage area of 27.5 square miles above the point of measurement, from April, 1890, to November, 1892, are given on page 70. The discharge of Genesee River at Mount Morris, above which point the drainage area is 1,070 square miles, from December, 1893, to November, 1896, is given on page 58. The table on pages 76 and 77 shows the quantity of water drawn from Hemlock Lake, also a tribu-

tary of Genesee River, and with a drainage area above the point of measurement of 43 square miles, for the water years 188[illegible] to 1884, inclusive. The table on page 75 gives a similar tabulation of water drawn from Skaneateles Lake for the months intervened from October, 1890, to November, 1897, inclusive. The tables on page 82 give the run-off of Hudson River as measured at Mechanicville, where the drainage area is 4,5[illegible] square miles, from October, 1887, to November, 1896, inclusive. The table on pages 83 to 85 gives the run-off of Croton River, as measured at the Croton dam, where the drainage area is 338 square miles, for the water years from 187[illegible] to 1886, inclusive. The table on page 72 presents measurements at Rochester in comparison with those at Mount Morris. So far as the author can learn, the foregoing include all the systematic measurements of streams, for considerable periods, thus far made in the State of New York, except those by John B. Jervis of the Madison and Eaton brooks in 1835, the results of which are presented in his report for that year to the canal commissioners.[1]

DISCHARGE MEASUREMENTS OF EATON AND MADISON BROOKS.

Eaton and Madison brooks, of which measurements were made by Mr. Jervis in 1835, are in the central-eastern part of Madison County and tributary to Chenango River. The drainage area of Eaton Brook is given by Mr. Jervis at 6,[illegible] acres, or 10.6 square miles, and that of Madison Brook as 6,000 acres, or 9.4 square miles.

Rainfall and run-off of Eaton Brook.

Month.	Rainfall.	Rainfall for 6,[illegible] acres.	Run-off from 6,[illegible] acres.	Percentage of run off to rainfall.
1835.	*Inches.*	*Cubic feet.*	*Cubic feet.*	
June	6.72	165,876,480	59,407,394	35.8
July	2.74	67,634,160	27,994,240	41.4
August	2.86	70,596,240	13,547,058	19.2
September	1.34	33,076,560	9,586,513	29.0
October	3.0	74,052,000	20,694,651	27.2
November	2.20	54,304,800	23,772,620	43.8
December	0.96	23,696,640	36,525,544	54.1
June to December, inclusive	19.82	489,236,880	191,528,020	39.2
June to October, inclusive	.....	411,235,440	131,229,856	31.9

[1] The measurements for short periods of several streams and of the water supply of Brooklyn are not overlooked in this statement, which is intended to apply to measurements extending over a year or more.

Rainfall and run-off of Madison Brook.

Month.	Rainfall.	Rainfall for 6,000 acres.	Run-off from 6,000 acres.	Percentage of run-off to rainfall.
1835.				
Snow of November-December, 1834, on ground	*Inches.*	*Cubic feet.* 87,120,000	*Cubic feet.*	
January	2.17	47,262,600	23,192,079	49.1
February	2.50	54,450,000	35,377,594	64.9
March	1.03	22,443,400	43,284,656	192.8
April	5.0	108,900,000	80,776,974	74.1
May	1.98	43,124,400	58,013,176	134.5
June	8.05	175,329,000	20,138,006	11.5
July	3.87	84,288,600	23,141,302	27.4
August	3.06	66,646,800	23,725,060	35.6
September	0.88	19,166,400	19,158,957	99.9
October	3.86	84,070,800	19,544,880	23.2
November	2.10	45,738,000	18,232,372	39.9
December	0.76	16,552,800	19,401,364	117.2
January to December, inclusive	35.26	855,092,800	383,986,420	44.9
January to May, inclusive		363,300,400	240,644,479	66.2
June to October, inclusive		429,501,600	105,708,205	24.6

The following statements in regard to these measurements a abstracted from Mr. Jervis's report:[1] From the Eaton Brook results appears that the average run-off from June to December, inclusiv was 39.2 per cent of the rainfall and from June to October, inclusiv 31.9 per cent of the rainfall. The minimum monthly run-off was August, which shows only 19.2 per cent of the rainfall. The rainfa in the month of June, 1835, on Eaton Brook was 6.72 inches and July 2.74 inches. The percentage of run-off to rainfall for June w 35.8, whereas for July it was 41.4, which would indicate that the bu of the June rain must have been at the end of the month.

[1] For Mr. Jervis's original report see Appendix F to Ann. Rept. Canal Com., 1835, Ass. I No. 65, pp. 55-60. Mr. Jervis's tables, with extracts from the report, are also quoted in the lowing documents:

(1) Report of F. C. Mills, chief engineer Gen. Val. Can., in Appendix D to Ann. Rept. Com., 1837, Ass. Doc. No. 80, p. 81.

(2) Report of W. H. Talcott, Res. Eng. Gen. Val. Can., 1840, Ass. Doc. No. 96, p. 51.

(3) Report of the Regents of the University, 1838, Sen. Doc. No. 52, pp. 208-211.

(4) Documentary History of the New York State Canals. By S. H. Sweet, Dep. State Eng Sur., 1863, Ass. Doc. No. 8, pp. 203-204.

From the measurements of Madison Brook it appears that in 1835 .e average run-off for the whole year, including the snow on the ·ound on January 1, was 44.9 per cent, or nearly one-half of the rain-.ll. Mr. Jervis points out that on account of the storage of the res-·voir Madison Brook record can not be taken for the summer months, ut that the year should be divided into two periods. For the first eriod he gives the results from January to May, inclusive, during hich the run-off was 66.2 per cent of the rainfall, and for the second ·om June to October, during which the run-off was 24.6 per cent of ıe rainfall. During the second period, June to October, inclusive, aton Brook gave a run-off of 31.9 per cent of the rainfall. Mr. ervis explains these different results by the different characters of ıe two districts drained. Eaton Brook Valley is very narrow and ıe area drained quite steep, with a very close-textured soil. Madi-on Brook Valley, on the other hand, is much wider, with easy slopes, nd the soil in a portion of it is more porous than that on Eaton Brook. Mr. Jervis concludes his discussion with the remark that Eaton Brook Valley would afford more than an average run-off over large district of country including the usual varities of soil, while Madison Brook would probably not differ materially from the general ·verage in this State.

In his documentary history of the New York State canals, which is ncluded in the annual report of the State engineer and surveyor for he fiscal year ending September 30, 1862, S. H. Sweet analyzes Mr. ervis's measurements of discharge of Eaton and Madison brooks and oints out several probable errors, especially in the Madison Brook ·sult, where, because the measurements indicate only what was actu-ly discharged through the sluice pipes each day instead of what ·ained off from the valley, he concludes that the real drainage of the adison Brook area in 1835 was about 0.518 of the rainfall, instead of 449, as given by Mr. Jervis. Inasmuch as the Eaton Brook and adison Brook measurements have only historical interest at the ·esent time, this branch of the subject is not here pursued at length.) far as can be learned, the measurements of these two streams by r. Jervis, in 1835, were the first systematic measurement of the run-f of streams in the United States. Geologically these streams lie in e horizon of the Hamilton shales.

DISCHARGE MEASUREMENTS OF OATKA CREEK.

The measurements of Oatka Creek, recorded in the following table, ·re made at the milldam in the south part of the village of Warsaw, Wyoming County. The dam was new, practically tight, and well apted for securing accurate results. Measurements were also made the outflow of the head race way leading from the dam for different ·vations of water on the dam, and a curve prepared from which the scharge of the race way was read off and added to the discharge over

the dam as computed by Francis's weir formula. It is believed that the results are accurate within a very small per cent.

Rainfall, run-off, evaporation, and mean temperature of Oatka Creek drainage area.

[In inches on the watershed.]

Months.	1890.				1891.				1892.			
	Rainfall.	Run-off.	Evaporation.	Mean temperature.	Rainfall.	Run-off.	Evaporation.	Mean temperature.	Rainfall.	Run-off.	Evaporation.	Mean temperature.
				°				°				°
December	3.33	.....	.....	37.0	2.61	0.97	.....	24.6	3.80	1.04	.....	30.9
January	4.36	.....	.....	32.7	4.12	2.62	.....	26.1	3.82	0.78	.....	21.0
February	3.63	.....	.....	31.5	4.67	3.40	.....	29.0	3.71	1.66	.....	24.9
March	2.72	.....	.....	29.5	3.70	2.87	.....	29.9	1.73	1.94	.....	28.1
April	2.82	2.17	.....	44.7	1.52	1.39	.....	45.7	1.04	2.21	.....	42.1
May	6.15	3.16	.....	52.8	1.60	0.63	.....	52.8	5.74	1.75	.....	50.9
	23.01	.....	.....	*38.0*	*18.22*	*11.88*	*6.34*	*34.7*	*19.84*	*9.60*	[illegible]	*31.8*
June	4.12	1.85	.....	67.4	4.01	0.44	.....	65.2	6.67	1.41	.....	65.2
July	3.18	0.38	.....	70.2	4.52	0.37	.....	63.9	4.18	2.06	.....	68.1
August	3.22	0.28	.....	66.8	4.25	0.25	.....	65.4	4.45	1.43	.....	65.1
	10.52	*2.51*	*8.01*	*68.1*	*12.78*	*1.06*	*11.72*	*64.8*	*15.30*	*4.90*	*10.40*	*67.[illegible]*
September	6.59	1.35	.....	60.3	1.72	0.46	.....	62.5	1.62	0.24	.....	59.0
October	4.52	2.27	.....	50.2	2.49	0.24	.....	46.6	2.19	0.33	.....	47.2
November	2.90	2.13	.....	39.3	2.91	0.41	.....	36.5	2.74	0.57	.....	35.0
	14.01	*5.75*	*8.26*	*49.9*	*7.12*	*1.11*	*6.01*	*48.5*	*6.55*	*1.14*	*5.41*	*47.[illegible]*
Year	47.54	.....	.....	48.5	38.12	14.05	24.07	45.7	41.69	15.42	26.27	45.4

The drainage area of Oatka Creek above Warsaw includes 27.5 square miles of rolling semimountainous country. The valley of the creek is deep cut, with numerous springs at the head waters. The drainage area is mostly deforested and in a high state of cultivation, the soil inclining to clay for a considerable portion. Geologically the stream lies in the rocks of the Portage formation, as developed in western New York. The run-off from this area may be taken as fairly typical of many small streams in western New York.

DISCHARGE MEASUREMENTS OF GENESEE RIVER.

The measurements of Genesee River proper, presented in the table on page 58, were made at the timber dam of the Mount Morris Hydraulic Power Company from September, 1893, to November, 1896, inclusive. The crest of this dam is quite irregular, and, in order to apply weir formulæ to it, an accurate profile was taken and the dam divided into a number of approximately level sections, with each section computed separately for various heights and advancing by 0.1 of a foot up to 10 feet. Working on this plan, the flow over the entire dam, which is 337 feet in length, was obtained by adding together the sums of the several sections at the corresponding heights and tabulating these. A gage graduated to 0.05 foot was erected on the river bridge

CANYON OF GENESEE RIVER BETWEEN MOUNT MORRIS AND PORTAGE.

hort distance away, with its zero level coinciding with the lowest nt of the dam. During ordinary stages of the river readings of this e were taken twice each day, but in time of high water, in order to ain the movement of floods as accurately as possible, readings e taken several times a day. In order to compute the flow readily urve was projected, embodying the data of the tabulation previ- ly referred to, and from which, with the given gage heights, the s in cubic feet per second could be quickly read off.

When the measurements were first begun, it was considered that formula $Q=1142\ H^{\frac{3}{2}}$ was best suited to the form of the dam, but er more careful consideration it was apparent that the results given this formula were somewhat in excess of the actual discharge, espe- lly for the low-water flows. Accordingly a weir was constructed ing the summer of 1896, at a point 2.5 miles above the Hydraulic ver Company's dam, where rock bottom clear across the river offered onvenient opportunity for such construction without heavy expense. s weir was made perfectly tight.

n order to correlate the measurements at the Mount Morris draulic Power Company's dam with those of the weir, two observa- s a day were taken at each place, nearly at the same time; that say, they were both taken by the same man, who passed imme- ely from the weir to the dam and vice versa. Observations on weir were obtained up to a head of 4 feet, and the corresponding harge computed with the proper allowance for velocity of oach, etc. The depths on the Hydraulic Power Company's dam sponding to the given depths on the weir were so plotted on the am as to give at once the relation between the flow at the weir the depth on the crest of the Hydraulic Power Company's dam. roceeding in this way the dam was accurately rated up to a dis- ge of 5,000 cubic feet per second. For discharges beyond 5,000 c feet per second the original determination has been used. An nsion of the plotted curves shows that some little distance above cubic feet per second discharge the results of the two methods substantially the same. The two curves crossed at the point of at 6,000 cubic feet per second discharge. For discharges above 00 or 15,000 cubic feet per second there is probably an error in results of from 5 to 10 per cent. Below 5,000 cubic feet per sec- it is believed that the results are now accurate within 2 or 3 per t. Francis's formula, $Q=3.33\ L\ H^{\frac{3}{2}}$ has been used for the weir putations.

he measurements taken previously to the construction of the weir the rating of the dam, as aforesaid, have all been corrected to form to the new determinations; hence all the data of the Genesee asurements of this table may be considered as accurate within the its stated.

Discharge measurements of Genesee River have also been kept for the last three years at Rochester, where the drainage area is about 2,400 square miles. From data obtained during the summer of 1895 it was apparent that these measurements also require a correction in order to give the approximate true flow of the stream during the period covered. Without going into detail it may be stated that the Rochester measurements in the following table give the corrected results, which are now probably accurate within from 5 to 10 per cent:

Comparison of the measurements of Genesee River at Rochester with those at Mount Morris for the water years 1894 to 1896, inclusive.

[In cubic feet per second, and with yearly means also in inches on the watershed.]

Month.	1894.			1895.			1896.		
	As per record.	Corrected.	Estimated from measurements at Mount Morris. *a*	As per record.	Corrected.	Estimated from measurements at Mount Morris. *a*	As per record.	Corrected.	Estimated from measurements at Mount Morris. *a*
December	3,914	3,914	4,797	1,459	1,100	1,256	1,839	1,700	2,70[illegible]
January	2,841	2,841	2,867	1,619	1,200	1,335	1,645	1,400	9[illegible]
February	2,584	2,584	1,954	977	700	495	2,702	2,702	2,0[illegible]
March	6,008	6,008	6,794	4,035	4,035	3,985	3,725	3,725	6,1[illegible]
April	5,646	5,646	7,172	3,083	3,083	4,257	7,623	7,623	7,12[illegible]
May	6 304	6,304	9,080	1,309	900	385	1,576	1,300	3[illegible]
	4,576	*4,576*	*5,477*	*2,099*	*1,848*	*1,958*	*3,191*	*3,054*	*3,[illegible]*
June	2,951	2,800	2,321	885	535	283	1,317	1,000	6[illegible]
July	1,055	732	292	645	390	232	854	645	5[illegible]
August	973	732	442	600	400	254	585	440	4[illegible]
	1,656	*1,426*	*1,003*	*728*	*440*	*256*	*977*	*692*	*5[illegible]*
September	1,664	1,500	1,963	407	250	221	324	240	3[illegible]
October	1,226	920	899	366	220	230	2,271	2,000	3,6[illegible]
November	1,782	1,600	1,729	834	500	993	993	745	1,7[illegible]
	1,571	*1,335*	*1,523*	*534*	*333*	*478*	*1,353*	*1,000*	*1,[illegible]*
Year	3,088	2,978	3,370	1,364	1,116	1,163	2,174	1,951	2,2[illegible]
Inches on watershed	19.20	18.35	19.38	8.48	6.41	6.67	12.48	11.20	12.[illegible]

a Increased in proportion to increased drainage area at Rochester.

As interesting data from Genesee River measurements, we may discuss the flood of May 20–23, 1894, at which time the approximate discharge of the stream at Mount Morris, from a drainage area of 1,070 square miles, was as follows:

Discharge of Genesee River at Mount Morris during the flood in May, 1894.

	Cubic feet per second
May 18, 7 a. m	600
May 18, 6 p. m	3,0[illegible]
May 19, 7 a. m	5,5[illegible]
May 19, 6 p. m	5,0[illegible]
May 20, 7 a. m	16,5[illegible]
May 20, 12 m	[illegible]

Discharge of Genesee River at M[illegible]

May 20, 6 p. m	[illegible]
May 21, 3.30 a. m	[illegible]
May 21, 7 a. m	[illegible]
May 21, 12 m	[illegible]
May 21, 6 p. m	[illegible]
May 22, 7 a. m	[illegible]
May 22, 12 m	[illegible]
May 22, 6 p. m	[illegible]
May 23, 7 a. m	[illegible]
May 23, 12 m	[illegible]
May 23, 6 p. m	[illegible]
May 24, 7 a. m	[illegible]

The total run-off from 7 a. m. of May [illegible] to 7 a. m. of May [illegible] was nearly 6,000,000,000 cubic feet.

On the morning of May 21 the flats in the broad level valley of

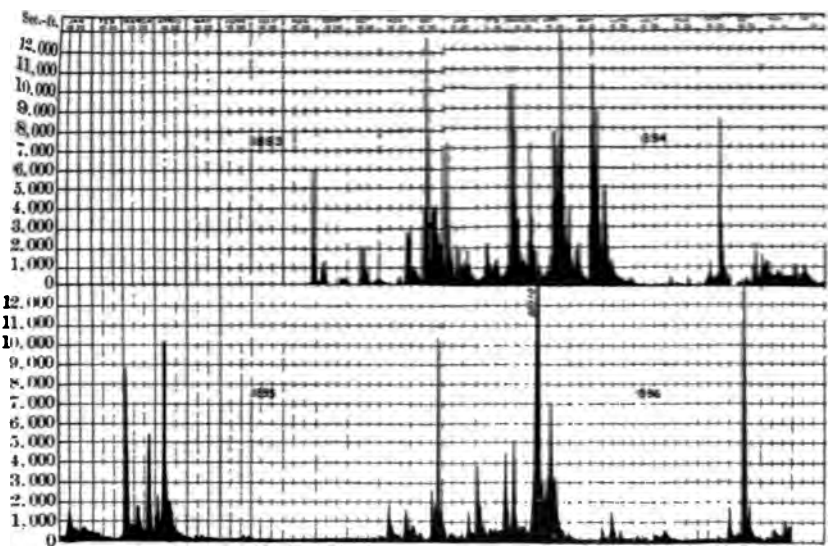


FIG. 2.—Discharge of Genesee River at Mount Morris, New York, 1883 to 1886

Genesee River and Canaseraga Creek, between Dansville, Mount Morris, and Rochester, and which have an area of from 60 to 80 square miles, were nearly flooded, in some localities to a depth of from 4 to 6 feet. On account of the large pondage by these flats, although the maximum run-off at Mount Morris was 42,000 cubic feet per second at 3.30 a. m. on the morning of May 21, at Rochester the maximum flow did not at any time exceed about 20,000 cubic feet per second. We have, then, a case where a large pondage has, by prolonging the time of run-off, modified a flood flow over 50 per cent. As further illustrating the effect of a large reservoir, or, what is the same thing, the effect of a large pond area in modifying the effect of an extreme

flood, reference may be made to fig. 3, in which, with time as abscissas and run-off as ordinates, the run-off record of Genesee River for May 18–24, 1894, has been plotted. The lower curve of that figure may be taken as representing approximately the law of the run-off of any generally distributed heavy rainfall on the catchment area of this stream. In making this statement it is not overlooked that flood flows at other seasons of the year may differ somewhat in their movement from that of May, 1894. Inasmuch as the rapidity and intensity of the run-off of any given stream depend largely upon the topography, the statement may be made that the general law of movement of floods in Genesee River is indicated by the lower curve of fig. 3. With this understanding we may assume any other run-off and construct the approximate curve by drawing it generally parallel to the curve of the actually observed case. In this way the upper curve of fig. 3, representing the curve of a flood one and one-half times greater than that of May, 1894, has been produced, slight irregularities of the lower curve having been neglected in projecting the upper one.

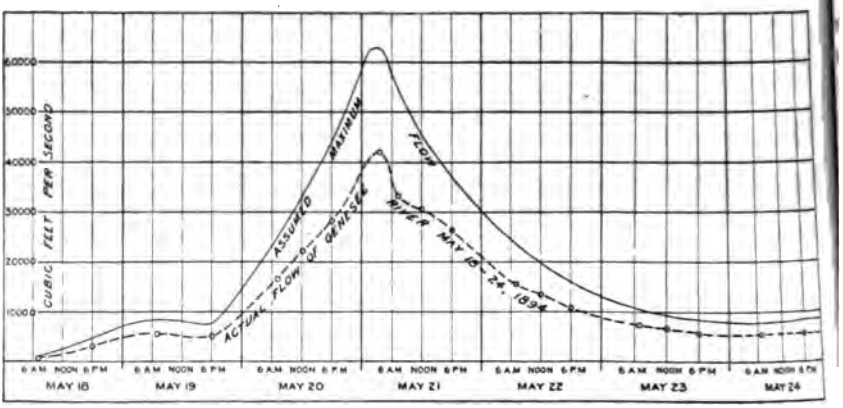


FIG. 3.—Flood flow of Genesee River, May 18–24, 1894.

A flood flow one and one-half times as great as that of May, 1894, which culminated in a maximum of about 42,000 cubic feet per second at 2.30 a. m. of May 21, gives a maximum of 63,000 cubic feet per second, the movement of which would be, under the assumptions, substantially as in the upper curve of fig. 3. As to the probability of a maximum flood flow of 63,000 cubic feet per second on the upper Genesee drainage area, the case of the neighboring Chemung River may be cited, where a flood flow of 67.1 cubic feet per second per square mile occurred in June, 1889. This figure applied to the upper Genesee would give a possible maximum run-off of 71,126 cubic feet per second.

Geologically, the drainage area of Genesee River above Mount Morris, the point of measurement, lies in the shales, sandstones, etc. of the Portage and Chemung groups. Its extreme head waters sout

Pennsylvania line issue from the lower Carboniferous. Gen-
he soils throughout the whole basin are heavy and tenacious,
g to clay. Their capacity for absorbing and retaining water
herefore, be considered as small.

DISCHARGE MEASUREMENTS OF HEMLOCK LAKE.

urements of the run-off of the Hemlock Lake drainage area for
er years 1880 to 1884, inclusive, were made by the Rochester
orks. Hemlock Lake lies at an elevation of 896 feet above
d has a length of 6.5 miles, with an average width of about
mile. The area of the surface at low water is 1,828 acres. The
ainage basin, including the area of the lake, is 27,554 acres, or
3 square miles. The shores are bold, and on the east side rise
ght of several hundred feet above the lake in a distance of 2
les. At the head of the lake there is a swamp of 118 acres,
y covered at high water.

utflow of the lake during the period covered by the measure-
ncluded in the following table may be considered as having
lace at three points: (1) At the natural outlet of the lake;
n artificial channel through which water was discharged at will
benefit of the millers on the outlet; and (3) through the con-
the Rochester waterworks. The run-offs given are the sums
e several outgoes. In order to determine the outflow of the
outlet, a weir was constructed and the discharge observed at
t heights of the lake surface. The discharge into the artificial
l was through submerged orifices of known dimensions, and has
mputed from standard formulæ for the discharge of such
, the size of the openings and the difference of level of water
s above and below being known.

lischarge of the conduit of the Rochester waterworks is as
ed from standard formulæ for discharge through pipes. Meas-
ts made by the author and others during the last few years
at the computed quantities passing through the conduit were
from correct. As a whole, it is believed that the Hemlock
sults are accurate within from 5 to 8 per cent.

Water drawn from Hemlock Lake for the water years 1880 to 1884, includ

[In inches on the watershed.]

Month.	1880.					1881.			
	Mean monthly elevation of lake surface.	Rainfall.	Water drawn.	Rainfall, less the water drawn.	Temperature.	Mean monthly elevation of lake surface.	Rainfall.	Water drawn.	Rainfall, less the water drawn.
December	−1.67	1.26	0.16	.....	a 31.4	−1.38	0.72	0.49	.....
January	−0.91	1.37	0.15	.....	a 25.5	−1.47	2.24	0.44	.....
February	−0.11	1.45	0.16	.....	a 27.3	−0.11	1.06	0.54	.....
March	+0.21	1.47	0.15	.....	a 31.8	+1.20	1.92	1.73	.....
April	+0.79	1.35	0.15	.....	a 45.7	+1.47	0.52	1.24	.....
May	+0.87	2.08	0.17	.....	71.8	+1.38	2.28	1.11	.....
	−0.14	*8.38*	*0.94*	*7.94*	*32.9*	*+0.12*	*8.71*	*5.55*	*3.16*
June	+0.45	1.66	0.28	.....	76.5	+1.08	3.13	0.76	.....
July	−0.15	1.98	0.41	.....	77.1	+0.58	2.71	0.43	.....
August	−0.70	3.46	0.35	.....	74.4	.....	0.95	0.50	.....
	−0.13	*7.05*	*1.19*	*5.26*	*76.0*	*+0.55*	*7.79*	*1.69*	*6.10*
September	−1.13	1.35	0.31	.....	69.7	−0.69	1.73	0.35	.....
October	−1.57	3.35	0.31	.....	52.4	−0.31	4.26	0.38	.....
November	−1.24	0.86	0.39	.....	37.1	−0.71	1.81	0.46	.....
	−1.31	*6.06*	*1.01*	*5.05*	*52.4*	*−0.74*	*7.77*	*1.14*	*6.63*
Year	−0.43	21.99	3.07	18.92	51.8	+0.04	24.27	8.38	15.89

a Interpolated from average of fifteen years.

Month.	1882.					1883.			
	Mean monthly elevation of lake surface.	Rainfall.	Water drawn.	Rainfall, less the water drawn.	Temperature.	Mean monthly elevation of lake surface.	Rainfall.	Water drawn.	Rainfall, less the water drawn.
December	−0.05	4.02	0.66	.....	39.8	−1.51	0.91	0.19	.....
January	+1.63	1.03	2.04	.....	29.4	−1.56	0.84	0.21	.....
February	+1.39	1.07	1.40	.....	37.0	+0.08	3.11	0.28	.....
March	+1.67	1.47	2.82	.....	38.7	+0.95	0.90	0.68	.....
April	+1.51	2.49	1.53	.....	48.5	+1.57	2.43	1.58	.....
May	+1.61	5.20	1.74	.....	57.4	+1.59	9.54	2.59	.....
	+1.29	*15.37*	*10.19*	*5.18*	*41.8*	*+0.18*	*17.73*	*5.53*	*12.20*
June	+1.45	2.31	1.85	.....	71.9	+1.38	4.52	1.65	.....
July	+0.81	1.42	0.62	.....	78.0	+1.29	2.13	1.08	.....
August	+0.15	2.17	0.41	.....	76.8	+0.64	2.86	0.45	.....
	+0.80	*5.90*	*2.88*	*3.02*	*75.6*	*+1.10*	*9.51*	*3.18*	*6.33*
September	−0.44	1.78	0.43	.....	69.4	+0.25	2.36	0.21	.....
October	−0.99	1.00	0.65	.....	61.4	+0.07	1.62	0.18	.....
November	−1.38	1.41	0.36	.....	41.8	+0.17	2.02	0.19	.....
	−0.94	*4.19*	*1.44*	*2.75*	*57.5*	*+0.16*	*6.00*	*0.58*	*5.42*
Year	+0.61	25.46	14.51	10.95	54.2	+0.41	33.24	9.29	23.95

awn from Hemlock Lake for the water years 1889 to 1884, etc.—Cont'd.

Month.	1884.				
	Mean monthly elevation of lake surface.	Rainfall.	Water drawn.	Rainfall, less the water drawn.	Temperature.
.....	−0.44	2.01	0.54	.....	34.0
.....	−0.46	1.78	1.01	.....	24.7
.....	−1.28	2.17	2.70	.....	30.2
.....	−1.47	3.18	2.71	.....	20.5
.....	−1.56	2.21	1.47	.....	42.7
.....	−1.62	3.30	1.69	.....	56.7
	−1.14	*14.65*	*10.12*	*5.53*	*36.5*
.....	−1.01	2.44	0.75	.....	70.4
.....	−0.58	3.98	0.37	.....	68.4
.....	−0.27	1.08	0.24	.....	70.8
	−0.62	*7.50*	*1.36*	*6.14*	*69.9*
.....	−0.26	2.24	0.71	.....	66.9
.....	−0.70	1.34	0.20	.....	52.3
.....	−1.17	1.01	0.18	.....	38.9
	−0.71	*4.59*	*1.09*	*3.50*	*52.7*
.....	−0.55	26.74	12.57	14.17	48.9

ainage area of Hemlock Lake is, as stated, 27,554 acres, and of the lake itself at the elevation ±0.0 is 1,828 acres; hence surface is 6.6 per cent of the total drainage area, or the drainage 5.1 times the area of the lake surface. On this basis 1 inch on e area is 15.1 inches on the lake. Taking into account these ts, it is clear that the data of the table give approximately ral run-off, although for exact figures corrections for actual s of lake surface at the beginning, as well as at the end of each uld be applied. On this point see the discussion on the mini- v of Hemlock Lake, on pages 92 and 93. Geologically, the Lake Basin proper is in the Hamilton and Marcellus shale, hills at the side rising into the rocks of the Portage group.

DISCHARGE MEASUREMENTS OF SKANEATELES LAKE.

easurements of the run-off of Skaneateles Lake drainage area, in the following table, have been made by the Syracuse ks over a dam at the foot of the lake or over a weir a short below since October, 1890. Previous to 1886 this lake was cipal feeder of the Jordan level of Erie Canal, but in that sco and Owasco lakes were also made feeders. The Skane- ke dam was reconstructed 9 feet high by the State in 1887, 893 was again rebuilt by the Syracuse water board with its 2 feet higher than the crest of the old dam. The area of the rface of Skaneateles Lake is 12.75 square miles, and the area atershed, including the area of the lake, is 73 square miles.

The elevation above tide is 867 feet. The lake lies in a deep valley with bold shores rising several hundred feet at either side. The figures given in the following table do not represent in any degree the natural run-off of this drainage area, but merely the water yield during the years indicated, in which there was large storage.

In March, 1895, the city of Syracuse began to draw water through its new conduit to Skaneateles Lake. Since that time the results given in the table are the quantity flowing in the outlet as measured on the weir located at Willow Glen plus the outflow through the conduit. Previous to March, 1895, the results are from measurements at the dam at the foot of the lake. The earlier results are possibly affected by errors of from 12 to 15 per cent, while the latter are probably accurate within from 2 to 5 per cent. Geologically the drainage basin of Skaneateles Lake is in the Hamilton group of rocks.

Water drawn from and monthly elevations of Skaneateles Lake for the months indicated for the water years 1890 to 1897, inclusive.

[Water drawn in inches on the watershed.]

Year, etc.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.
1890:												
Mean lake surface	−1.92	−2.08	−2.25	+0.33	+0.33	+0.25	+0.37	+0.50	−0.42	−1.33	−1.08	−1.1
Water drawn	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.59	1.6
1891:												
Mean lake surface	−1.00	−1.50	−1.00	+0.16	+0.50	0.0	−0.75	−1.42	−2.21	−3.25	−4.12	−5.2
Water drawn	1.62	1.63	1.52	2.42	2.57	a 1.90	1.23	1.33	1.93	1.79	1.61	1.2
1892:												
Mean lake surface	−5.79	−5.50	.....	−5.0	−3.67	−1.83	−0.67	−0.33	−0.67	−1.08	−1.83	−2.5
Water drawn	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.35	1.24	1.1
1893:												
Mean lake surface	−2.56	−2.71	−2.92	−2.75	−1.33	0.0	+0.33	−0.67	−1.42	−2.17	−3.00	−3.8
Water drawn	1.09	0.87	0.87	1.10	1.18	3.06	2.10	1.98	[1] 1.53	1.08	1.58	1.5
1894:												
Mean lake surface	−4.75	−4.48	.....	.....	.....	−2.50	−1.27	+0.25	−0.06	−0.88	−1.00	−1.2
Water drawn	1.38	0.86	.....	.....	.....	0.26	0.15	0.57	1.07	1.39	1.46	1.4
1895:												
Mean lake surface	−1.58	−1.20	−1.75	−0.37	−0.17	+0.14	−0.40	−0.92	−1.67	−2.18	−3.12	−4.1
Water drawn	0.32	0.24	0.22	1.29	2.55	2.14	1.64	1.77	1.87	1.64	1.85	1.7
1896:												
Mean lake surface	−4.56	−4.33	−4.60	−4.08	−2.42	−1.35	−1.58	−1.94	−2.23	−2.94	−3.42	−3.8
Water drawn	1.58	1.51	1.46	1.61	1.50	1.59	1.61	1.51	1.50	1.51	1.50	1.3
1897:												
Mean lake surface	−3.92	−4.12	−4.29	−4.56	−3.46	−2.67	−2.50	−2.65	.....	.....	.....	.....
Water drawn	1.35	1.25	1.08	1.06	0.26	1.12	1.34	1.45	2.21	1.41	1.61	1.4

a Interpolated. No record. Mean of preceding and following months used.

DISCHARGE MEASUREMENTS OF HUDSON RIVER.

leasurements of the flow of Hudson River have been made over dam of the Duncan Company, at Mechanicville. In 1887 this npany began daily measurement of the amount of water flowing in dson River at their mill.[1] With the exception of one or two days s record has been kept for every working day since October 1, 1887. record has also been kept of the number, size, and kind of turbine ter wheel in use for the same period. The Duncan Company placed this material at the disposal of the survey of the Upper Hudson lley, of which the author has had charge, thus enabling him to npute the mean daily flow of the river for each working day from tober 1, 1887, to November 30, 1896. The flow of Sundays and holys, when no observations were taken, has been assumed as a mean :ween the preceding Saturday and the following Monday, etc. The m is a substantial structure of masonry 16 feet high, with a length 794 feet between the abutments. The crest is stated by John R. lley, the constructing engineer, to be perfectly level, and from all it can be learned it appears that the daily observations have been ten with such care as to leave no reason for doubting that this is a rly accurate exhibit of the daily flow of the stream for the period ered. This record is therefore considered to be accurate within m 5 to 8 per cent.

The greatest depth on this dam in the nine-year period, 1888 to 1896, lusive, occurred May 5, 1893, when the gage showed a depth of 3 feet and the mean flow of the day was over 53,000 cubic feet per ond. The drainage area of Hudson River above the Mechanice dam is taken at 4,500 square miles.

Experience in flows over dams of this length and with depths as at as from 7 to 8 feet is as yet rather limited in this country, and question was raised as to the best method of computing the disrge for a case like the one under discussion. The engineers of the tish Government in India have had, in connection with their large gation works, perhaps more experience in this class of measureit than all others combined, and the formulæ used by them appear re rational in form than those commonly used in the United States such computations, and after some study it was decided to use se. As many American engineers may not be familiar with these mulæ they are here reproduced. They take the following form:

$$Q = \tfrac{2}{3} L C \sqrt{2 g} l^{3}, \qquad (1)$$

which—

= the discharge over a thin-edged clear overfall, in cubic feet per second,

= the length of the dam in linear feet,

[1] Annual Report of the State Engineer and Surveyor of New York, 1895, p. 104.

C = coefficient depending for its value on d,
g = acceleration of gravity = 32.2,
d = depth on crest, in linear feet.

Equation (1) may also take the form—

$$Q = 5.35\ L\ C\ \sqrt{d^3}. \tag{2}$$

To find C for different values of d, we have—

$$C = 1 - \left(\frac{0.04\ (34.6 + d)}{4}\right). \tag{3}$$

This gives a series of values of C corresponding to d. For instance, for $d = 0.25$ foot, $C = 0.651$; for $d = 0.50$ foot, $C = 0.649$, and so on.

For a wide-crested dam the coefficient is further modified to suit the actual width of the crest. For this we have given the expression—

$$C' = C - \left(\frac{0.025\ C\ (B + 1)}{1 + d}\right), \tag{4}$$

in which—

B = the width of the crest in linear feet;
C = the coefficient for a thin-edged weir, corresponding to a depth d, as per equation (3), and
C′ = the adjusted coefficient corresponding to a given breadth B and a depth d.[1]

In the case of the Mechanicville dam we have a stone crest 7 feet in width and slightly inclined upstream. The width of the river a short distance above the dam is considerably over 800 feet; the depth for some distance back is from 16 to 20 feet. In order to avoid a correction for velocity of approach, a crest was assumed 5 feet wide and values of C′ were computed on that basis.

Having obtained values of C′ for $d = 0.25, 0.50, 0.75, 1.00, 1.25, 1.50, 1.75$ feet, and so on up to 8 feet, corresponding values of Q were computed and plotted at a large scale as a curve with values of d as abscissas and the corresponding flows as ordinates. From this curve intermediate values of Q have been read off.

[1] The method of deducing equations (3) and (4) may be found in Mullin's Irrigation Manual, 1890, pp. 11, 12, 138, 139, 171, 172.

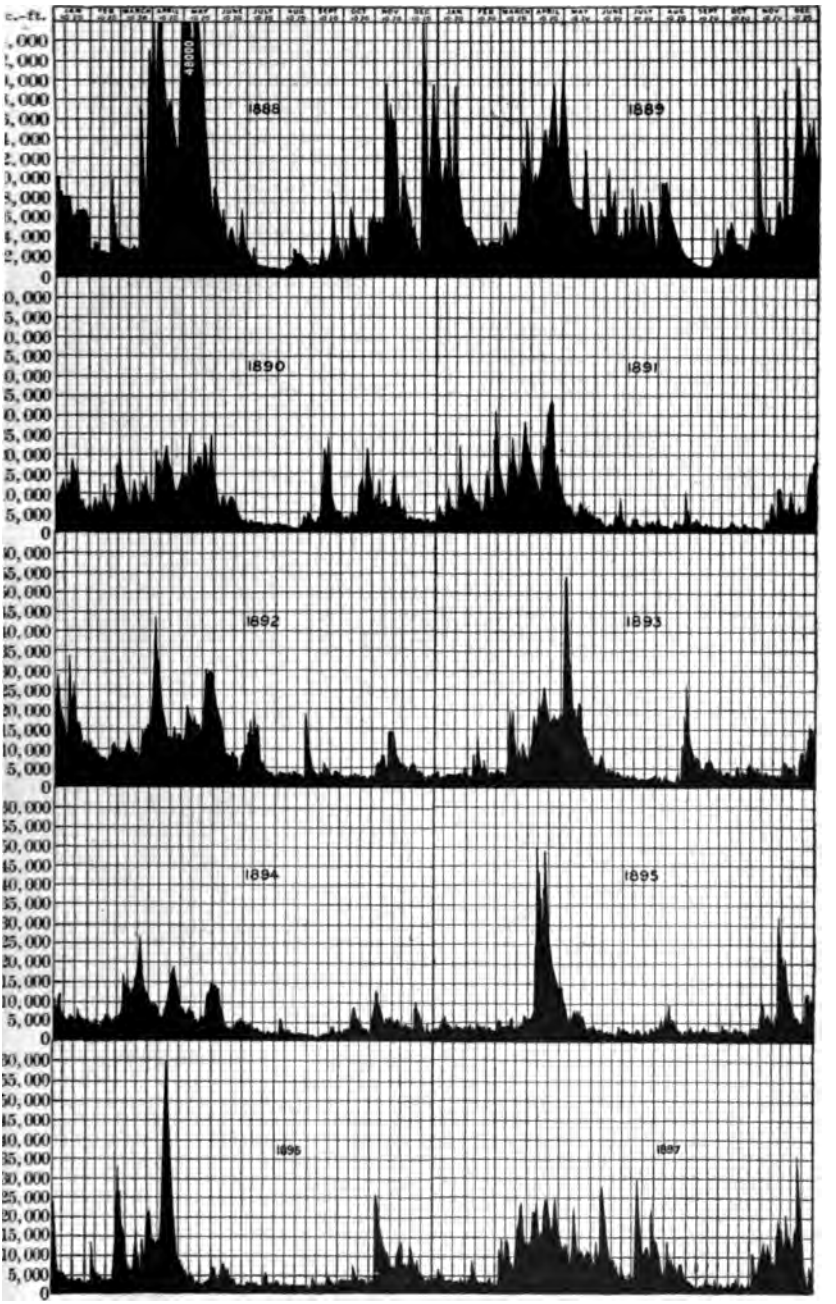


FIG. 4.—Discharge of Hudson River at Mechanicville, New York, 1888 to 1897.

Run-off of Hudson River at Mechanicville from October, 1887, to November, 1896, inclusive. (*a*)

[In cubic feet per second.]

Month.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.
December	.....	8,018	10,014	13,226	3,244	8,577	4,031	7,217	4,367	10,[illegible]
January	.....	6,367	10,983	11,272	8,284	18,857	3,192	6,757	3,876	6,[illegible]
February	.....	3,714	3,790	7,913	11,664	9,203	4,805	4,836	3,543	4,[illegible]
March	.....	6,845	8,280	11,129	17,736	10,929	8,250	14,738	4,204	13,[illegible]
April	.....	21,200	13,690	15,053	20,021	21,554	17,889	11,135	23,822	24,[illegible]
May	.....	21,420	8,871	17,931	5,533	19,622	22,285	7,566	6,850	4,[illegible]
Mean	.....	*11,289*	*9,337*	*12,821*	*11,021*	*14,831*	*10,114*	*8,759*	*7,759*	*10,[illegible]*
June	.....	4,917	6,869	7,392	3,200	12,395	4,801	7,097	2,816	4,[illegible]
July	.....	1,537	5,727	1,950	2,337	9,287	2,521	3,168	2,559	2,[illegible]
August	.....	1,725	4,272	2,019	2,666	5,485	5,005	2,456	3,901	2,[illegible]
Mean	.....	*2,703*	*5,718*	*3,748*	*2,957*	*9,019*	*4,102*	*4,209*	*3,095*	*3,[illegible]*
September	.....	2,851	1,963	8,844	2,040	4,448	6,870	1,889	2,629	2,[illegible]
October	2,365	4,608	3,740	9,215	1,472	2,819	3,865	3,649	2,631	4,[illegible]
November	4,190	10,642	7,888	9,121	4,088	7,604	3,639	6,379	8,421	11,[illegible]
Mean	.....	*6,018*	*4,522*	*9,061*	*2,521*	*4,934*	*4,781*	*3,969*	*4,539*	*6,[illegible]*
Yearly mean	.....	7,820	7,197	9,597	6,867	10,909	7,271	6,418	5,780	7,[illegible]

[In inches on the watershed.]

December	.....	2.05	2.57	3.39	0.83	2.27	1.03	1.85	1.12	2.[illegible]
January	.....	1.62	2.81	2.89	2.12	4.83	0.82	1.73	0.99	1.7[illegible]
February	.....	0.89	0.88	1.83	2.70	2.22	1.09	1.12	0.82	1.1[illegible]
March	.....	1.75	2.12	2.85	4.55	2.80	2.11	3.78	1.08	3.[illegible]
April	.....	5.26	3.39	3.73	4.97	5.35	4.44	2.76	5.91	6.[illegible]
May	.....	5.49	2.27	4.59	1.42	5.03	5.71	1.94	1.76	1.1[illegible]
Total	.....	*17.06*	*14.04*	*19.28*	*16.59*	*22.50*	*15.20*	*13.18*	*11.68*	*16.[illegible]*
June	.....	1.22	1.70	1.83	0.79	3.08	1.19	1.76	0.70	1.1[illegible]
July	.....	0.39	1.47	0.50	0.60	2.38	0.65	0.81	0.66	0.7[illegible]
August	.....	0.44	1.09	0.52	0.68	1.41	1.28	0.63	1.00	0.[illegible]
Total	.....	*2.05*	*4.26*	*2.85*	*2.07*	*6.87*	*3.12*	*3.20*	*2.36*	*3.[illegible]*
September	.....	0.71	0.49	2.19	0.51	1.10	1.70	0.47	0.65	0.7[illegible]
October	0.61	1.18	0.96	2.36	0.38	0.72	0.99	0.94	0.69	1.[illegible]
November	1.04	2.64	1.96	2.26	1.01	1.89	0.90	1.58	2.08	2.[illegible]
Total	.....	*4.53*	*3.41*	*6.81*	*1.90*	*3.71*	*3.59*	*2.99*	*3.42*	*4.[illegible]*
Yearly total	.....	23.64	21.71	28.56	20.56	33.08	21.90	19.37	17.46	23.[illegible]

a Annual Report of the State Engineer and Surveyor of New York, 1895, pp. 107, 120.

DISCHARGE MEASUREMENTS OF CROTON RIVER.

The record of the run-off of Croton River as measured at the old Croton dam for the water years 1870 to 1896, inclusive, is given in the table on pages 83 to 85. The watershed of the Croton consists of a broken, hilly country with its surface soil composed principally of sand and gravel. Clay, hardpan, and peat, while found in a few localities, are for the whole area only present to a limited extent. The rock formation consists generally of gneiss, although strata of limestone, some micaceous and talcose slates, with veins of granite, serpentine, and

ore, occur in a few places. The drainage area lies almost entirely e State of New York, only a small portion being in Connecticut. ounts to about 338 square miles above the old Croton dam and 0 square miles above the new Croton dam under construction. main river is formed by three branches, known, respectively, as Middle, and West branches, which, rising in the southern part utchess County, flow south through Putnam County and unite its south boundary. The river then flows across Westchester ty to Hudson River, into which it empties at Croton Point, about iles north of the city of New York. The principal tributaries from East, Middle, and West branches are Kisko, Titicus, Cross, Muscoot rivers. The monthly and annual rainfall of the Croton rshed, as well as the run-off of the water years, from 1870 to 1896, sive, are given in the following table. The average annual rain- or this period was 48.10 inches and the run-off 24.65 inches.[1]

all and run-off of Croton River drainage area from 1870 to 1896, inclusive.

[In inches on the watershed.]

nth.	1870.		1871.		1872.		1873.		1874.		1875.	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
ber	5.96	3.07	1.49	0.64	2.59	2.11	3.68	1.45	4.13	3.38	1.78	0.98
y	4.51	3.99	3.80	0.59	1.44	2.08	5.66	4.29	6.96	8.22	2.74	0.65
ary	6.40	4.28	3.81	2.21	1.22	1.25	3.09	1.72	2.78	2.79	3.47	4.09
	3.80	3.56	4.27	3.52	2.59	1.75	3.08	4.03	1.57	3.03	4.99	3.24
	5.45	4.11	3.01	2.02	3.04	3.11	3.77	7.12	6.31	3.63	3.04	5.58
	2.30	1.86	3.45	2.06	3.69	1.29	2.91	2.19	1.99	3.19	1.08	1.86
	28.42	*20.87*	*19.83*	*11.04*	*14.57*	*11.59*	*22.19*	*20.80*	*23.74*	*24.24*	*17.10*	*16.40*
	2.06	0.83	5.73	1.43	4.00	1.22	0.71	0.54	3.57	0.92	3.02	0.59
	3.43	0.51	5.07	0.73	4.34	0.61	2.21	0.49	5.98	1.43	3.10	0.58
t	5.10	0.51	5.24	0.85	5.99	1.64	5.73	0.71	2.75	0.89	10.33	5.80
	10.59	*1.85*	*16.04*	*3.01*	*14.33*	*3.47*	*8.65*	*1.74*	*12.30*	*3.24*	*16.45*	*6.97*
ber	2.85	0.35	1.44	0.63	3.69	1.25	3.73	0.52	3.56	0.58	2.11	0.90
r	4.73	0.42	6.18	1.92	2.15	1.13	5.13	1.45	2.40	0.81	3.61	0.85
ber	2.51	0.62	4.35	3.41	4.91	2.67	3.72	1.81	2.72	0.74	4.61	2.05
	10.09	*1.39*	*11.97*	*5.96*	*10.75*	*5.05*	*12.58*	*3.78*	*8.68*	*2.13*	*10.33*	*3.80*
'otal	49.10	24.11	47.84	20.01	39.65	20.11	43.42	26.32	44.72	29.61	43.88	27.17

Wegmam's History of the Water Supply of the City of New York, Chap. IX, The Croton hed.

Rainfall and run-off of Croton River drainage area from 1870 to 1896, etc.—Cont'd.

Month.	1876.		1877.		1878.		1879.		1880.		1881.	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
December	1.56	1.84	2.35	1.19	1.52	1.97	8.74	7.23	4.26	2.03	2.49	[illegible]
January	1.42	1.59	2.68	0.84	4.49	2.85	2.52	1.45	4.00	2.75	4.19	[illegible]
February	4.91	3.65	0.80	1.55	3.65	3.92	2.85	2.77	2.92	2.99	5.28	[illegible]
March	6.33	7.16	7.66	6.97	3.10	3.89	4.96	4.30	4.51	3.01	6.14	[illegible]
April	4.43	6.39	2.35	3.02	2.85	1.69	5.10	5.12	3.99	2.09	1.67	[illegible]
May	3.99	2.03	0.85	0.89	4.97	1.57	2.45	1.77	1.17	0.98	3.74	[illegible]
	22.64	*22.66*	*16.69*	*14.46*	*20.58*	*15.89*	*26.62*	*22.64*	*20.85*	*13.85*	*23.51*	[illegible]
June	2.52	0.71	4.95	0.62	4.65	1.52	5.29	0.94	1.28	0.52	5.72	[illegible]
July	3.42	0.55	4.65	0.51	4.28	0.74	5.95	0.73	5.65	0.54	2.45	[illegible]
August	1.20	0.50	2.54	0.49	2.66	0.68	5.83	1.48	3.60	0.52	1.71	[illegible]
	7.14	*1.76*	*12.14*	*1.62*	*11.59*	*2.94*	*17.07*	*3.15*	*10.53*	*1.58*	*9.88*	[illegible]
September	5.21	0.37	1.49	0.34	6.61	2.13	3.43	1.09	2.69	0.50	0.75	[illegible]
October	1.50	0.38	8.38	1.14	3.78	0.93	0.95	0.67	3.25	0.51	3.65	[illegible]
November	3.40	0.71	8.16	4.18	4.36	2.09	2.49	0.82	2.97	0.57	4.50	[illegible]
	10.11	*1.46*	*18.03*	*5.66*	*14.75*	*5.15*	*6.87*	*2.58*	*8.91*	*1.58*	*8.90*	[illegible]
Total	39.89	25.88	46.86	21.74	46.92	23.98	50.56	28.37	40.29	17.01	42.29	[illegible]

Month.	1882.		1883.		1884.		1885.		1886.		1887.	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
December	6.53	1.73	2.68	1.29	3.45	0.64	7.34	3.78	3.84	2.09	4.29	[illegible]
January	4.41	2.40	2.80	1.06	5.07	2.14	5.59	4.12	5.24	3.42	5.68	[illegible]
February	5.96	4.23	5.21	3.81	6.31	4.95	4.66	2.44	5.20	4.89	6.01	[illegible]
March	4.58	4.74	1.67	3.03	4.82	5.01	1.29	2.05	3.86	2.50	3.60	[illegible]
April	1.36	1.43	3.94	2.73	2.96	3.00	2.09	2.62	3.61	4.51	3.47	[illegible]
May	6.30	2.20	2.86	1.38	4.33	1.91	2.44	1.58	4.54	2.15	0.32	[illegible]
	29.14	*16.73*	*19.16*	*13.30*	*26.94*	*17.65*	*23.41*	*16.59*	*26.29*	*19.56*	[illegible]	[illegible]
June	3.04	1.70	5.64	0.63	2.04	0.70	1.19	0.59	3.09	0.78	7.70	[illegible]
July	3.63	0.69	4.26	0.52	6.54	0.81	5.27	0.52	4.40	0.61	13.32	[illegible]
August	3.92	0.52	2.09	0.52	4.50	1.18	7.35	0.58	3.21	0.58	7.06	[illegible]
	10.59	*2.91*	*11.99*	*1.67*	*13.08*	*2.69*	*13.81*	*1.69*	*10.71*	*1.97*	*28.08*	[illegible]
September	14.33	3.25	2.45	0.51	1.69	0.74	1.09	0.42	2.30	0.51	2.00	[illegible]
October	3.33	2.27	6.99	0.66	3.74	0.57	5.19	0.55	2.28	0.52	3.12	[illegible]
November	1.66	0.96	1.79	0.69	4.37	0.98	5.99	2.19	5.57	0.88	2.69	[illegible]
	19.32	*6.48*	*11.23*	*1.86*	*9.80*	*2.29*	*12.27*	*3.16*	*10.15*	*1.91*	*7.81*	[illegible]
Total	59.05	26.12	42.38	16.83	49.82	22.63	49.49	21.44	47.15	23.44	59.26	[illegible]

nd run-off of Croton River drainage area from 1870 to 1896, etc.—Cont'd.

	1888.		1889.		1890.		1891.		1892.		1893.	
	Rain-fall.	Run-off.	Rain-fall.	Run-off.	Rain-fall.	Run-off.	Rain-fall.	Run-off.	Rain-fall.	Run-off.	Rain-fall.	Run-off.
...	6.71	2.48	6.13	5.26	2.94	4.55	3.71	1.65	5.65	1.64	1.11	1.48
...	5.56	4.01	5.14	4.41	2.03	2.34	9.76	6.84	5.95	5.07	3.29	1.70
...	5.07	4.95	2.33	2.36	4.94	3.19	6.02	5.73	1.22	1.54	4.60	3.27
...	6.44	4.69	1.86	2.10	5.66	4.72	3.36	4.37	2.90	2.10	4.52	7.21
...	2.68	4.85	4.42	2.58	3.03	3.29	3.77	2.90	1.08	1.42	3.55	3.57
...	6.27	2.67	3.22	1.76	5.74	2.73	1.36	0.89	5.74	1.02	8.18	6.06
	32.73	*23.65*	*23.10*	*18.47*	*24.34*	*20.82*	*27.98*	*22.38*	*22.54*	*13.39*	*25.25*	*23.29*
...	2.00	1.39	4.51	1.43	3.56	1.53	1.81	0.63	3.84	1.15	2.43	0.96
...	2.43	0.67	7.74	1.63	5.46	0.71	2.97	0.40	5.05	0.70	2.38	0.40
...	6.87	1.11	2.90	4.03	4.70	0.59	5.61	0.31	6.12	0.90	7.06	0.72
	11.30	*3.17*	*15.15*	*7.09*	*13.72*	*2.83*	*10.39*	*1.34*	*15.01*	*2.75*	*11.87*	*2.09*
...	10.77	3.11	6.13	2.27	6.86	2.04	1.87	0.38	2.65	0.51	2.65	0.53
...	4.80	2.58	4.85	1.93	7.63	3.36	2.15	0.41	0.92	0.17	6.42	1.17
...	4.49	3.04	8.45	5.29	1.12	2.09	3.86	0.64	7.85	1.58	3.32	2.08
	20.06	*8.73*	*19.43*	*9.49*	*15.61*	*7.49*	*7.88*	*1.43*	*11.42*	*2.26*	*12.39*	*3.78*
...	64.09	35.55	57.68	35.05	53.67	31.14	46.25	25.15	48.97	18.40	49.51	29.15

h.	1894.		1895.		1896.		Mean.	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
.....	5.34	4.12	4.43	2.57	4.88	1.30	.....	.....
.....	3.40	1.77	3.63	3.42	1.52	2.06	.....	.....
.....	5.01	2.15	3.34	1.04	6.65	4.33	.....	.....
.....	1.62	5.21	1.88	3.87	8.20	7.90	.....	.....
.....	3.07	2.44	5.63	4.22	0.96	3.05	.....	.....
.....	6.67	1.88	2.41	1.33	3.09	0.87	.....	.....
	25.11	*17.57*	*21.32*	*16.45*	*25.30*	*19.51*	*23.44*	*18.00*
.....	1.69	1.48	1.89	0.49	3.79	0.97	.....	.....
.....	1.75	0.45	3.95	0.44	3.98	0.86	.....	.....
.....	1.45	0.71	3.10	0.62	4.56	0.76	.....	.....
	4.89	*2.64*	*8.94*	*1.55*	*12.33*	*2.59*	*12.54*	*2.90*
.....	7.49	0.57	1.16	0.09	6.50	0.65	.....	.....
.....	5.94	0.75	3.55	0.40	2.17	0.88	.....	.....
.....	4.44	3.37	2.91	0.82	3.96	1.73	.....	.....
	17.87	*4.69*	*7.62*	*1.31*	*12.63*	*3.26*	*12.12*	*3.14*
.....	47.87	24.90	37.88	19.31	50.26	25.36	48.10	24.65

off as given is stated by A. Fteley, chief engineer of new Croton aqueduct, to have ted as far as necessary for the storage, and accordingly represents approximately l run-off of the stream.

Evaporation from the Croton River watershed as given by differences between rainfall and run-off in preceding table.

[Inches on watershed.]

Year.	December to May.	June to August.	September to November.	Total.	Year.	December to May.	June to August.	September to November.	Total.
1870	7.55	8.74	8.70	24.99	1885	6.82	12.12	9.11	[illegible]
1871	8.79	13.03	6.01	27.83	1886	6.73	8.74	8.24	[illegible]
1872	2.98	10.86	5.70	19.54	1887	6.21	20.73	4.93	[illegible]
1873	1.39	6.91	8.80	17.10	1888	9.08	8.13	11.33	[illegible]
1874	*a* 0.50	9.06	6.55	15.11	1889	4.63	8.06	9.94	[illegible]
1875	0.70	9.48	6.53	16.71	1890	3.52	10.89	8.12	[illegible]
1876	*a* 0.02	5.38	8.65	14.01	1891	5.60	9.05	6.45	[illegible]
1877	2.23	10.52	12.37	25.12	1892	9.15	12.26	9.16	[illegible]
1878	4.69	8.65	9.60	22.94	1893	1.90	9.79	8.61	[illegible]
1879	3.98	13.92	4.29	22.19	1894	7.54	2.25	13.18	[illegible]
1880	7.00	8.95	7.33	23.28	1895	4.87	7.39	6.31	[illegible]
1881	8.53	7.11	7.39	23.03	1896	5.79	9.74	9.37	[illegible]
1882	12.41	7.68	12.84	32.93	Mean	5.44	9.64	8.36	[illegible]
1883	5.86	10.32	9.37	25.55					
1884	9.29	10.39	7.51	27.19					

a During this period the run-off exceeded rainfall.

This stream is an exceedingly good water yielder. The minimum yield for a complete water year for the whole period from 1870 to 1896 was in 1883, in which water year, from December to November, inclusive, the total run-off was 16.83 inches.

The Croton watershed contains 31 lakes and ponds, many of which have been utilized as natural storage basins by constructing dams at their outlets. The following tabulation gives the entire natural and artificial storage, either actually carried out or now under construction, in the Croton watershed:

Storage capacity in the Croton watershed.

	U. S. gallons.
Boyds Corners reservoir	2,727,000,000
Middle Branch reservoir	4,004,000,000
Lake Mahopac *a*	575,000,000
Lake Kirk *a*	565,000,000
Lake Glenida *a*	165,000,000
Lake Gilead *a*	380,000,000
Lake Waccabuc *a*	200,000,000
Lake Tonnetta *a*	50,000,000
Barretts Pond *a*	170,000,000
China Pond *a*	105,000,000
White Pond *a*	100,000,000
Pine Pond *a*	75,000,000
Long Pond *a*	60,000,000

a The lakes and ponds marked thus are owned by the city. Those not marked are city reservoirs.

Storage capacity in the Croton watershed—Continued.

	U. S. gallons.
ach Pond *a*	230,000,000
oss Pond *a*	110,000,000
ines Pond *a*	25,000,000
st Branch reservoir	9,028,000,000
ticus reservoir	7,000,000,000
ramel reservoir	9,000,000,000
w Croton reservoir	32,000,000,000
awalk reservoir	7,000,000,000
Total storage	73,569,000,000

The lakes and ponds marked thus are owned by the city. Those not marked are city rvoirs.

The drainage area above the new Croton dam now constructing is) square miles. It is considered by the Croton aqueduct officials t the storage afforded by this reservoir system will furnish a daily pply of at least 280,000,000 gallons. At this rate the utilization from s drainage area of 360 square miles will become 778,000 gallons per are mile per day, or 1.20 cubic feet per square mile per second.

AXIMUM AND MINIMUM FLOW OF STREAMS IN NEW YORK.

The data relating to floods in Genesee River, given on pages 72 to as well as the following facts, may be taken as indicating some of maximum flows of streams in New York.

FLOODS IN CHEMUNG RIVER.

evere floods have occurred in this stream several times during the torical period, the most severe being the great flood of June, 1889, ich caused serious damage to property at Elmira and Corning. emung River is formed by the junction of Tioga and Cohocton ers at Painted Post, a few miles above Corning, the principal tribu- y of the Tioga in this State being the Canisteo. Tioga River rises r Blossburg, in Tioga County, Pennsylvania, in an elevated region m 1,500 to 2,500 feet above tide. The descent from the extreme d waters near the Fall Brook Coal Company's mines to Blossburg t the rate of about 22 feet per mile, after which it descends at the e of about 11 feet to the mile. The streams tributary to the Tioga also very rapid; they flow mainly through deep, narrow rock val- s, with their heads generally at an elevation of nearly 2,000 feet ve tide. Recently the hill slopes have been largely denuded of ber, thus permitting a rapid descent of the rainfall or melted snow. nce it results that Tioga River not only naturally rises quickly, its freshet flows have very high velocities. Canisteo River, join- the Tioga from the west, has an average slope of about 5.5 feet mile. The slope of the Chemung from Painted Post to Elmira is

at the rate of 5.9 feet per mile; thence through the city of Elmira for 3 miles at the rate of 3.5 feet per mile; from Elmira to Chemung, 5.5 feet per mile, and thence to Athens, where it joins the main East Branch of the Susquehanna, 3.4 feet per mile. Cohocton River is also a stream of comparatively low slope.

The foregoing facts indicate that the floods will be slower in rising on Canisteo and Cohocton rivers than on the Tioga. In a general storm, where other conditions are equal, the first flood water to reach Corning and Elmira will come from the Tioga, to be followed later by large flows from the Canisteo and the Cohocton. The areas drained by these streams are approximately as follows:

Areas drained by tributaries of Chemung River.

	Square miles
Tioga, aside from the Canisteo	750
Canisteo	780
Cohocton	425
Chemung above Elmira	100
Total	2,055

On May 31, 1889, the region tributary to Chemung River above Elmira was visited by a phenomenally heavy rainfall, amounting in many places to nearly 10 inches. The center of this downpour was located about 10 miles south and 15 miles west of Elmira. At Elmira the rainfall was not unusual, 1.5 inches being recorded from 8 p. m. of May 31 to 7 a. m. of June 1; but at Wellsboro, 36 miles southwesterly from Elmira, the total precipitation was 9.8 inches, of which 7.45 inches occurred after 9 p. m. of May 31 and before 7 a. m. of June 1. At South Canisteo, 45 miles westerly from Elmira, a total fall of 6.25 inches was recorded, of which 4.5 inches fell between midnight and 3 a. m. of June 1. Farther up the valley 6 inches were measured between the same hours. At Painted Post a total fall of about 8 inches was reported. At Savonia, on the Cohocton, 5 inches fell, but the fall grew gradually less to the north. At a number of points to the south and southwest rainfalls of from 6 to 8 inches were recorded for May 31, heavy rains occurring as far south as Virginia.

It will be noticed from the preceding statement of the rainfall of May 31 and June 1, 1889, that the heaviest precipitation was practically at the same time over the entire watershed. The following indicates the heights of the flood wave at several points: At Tioga the river was highest about 6.30 a. m. on June 1; Canisteo River was at its highest a little before noon of June 1; at Painted Post the local creeks reached their highest points at 5 a. m., and the Tioga began to rise rapidly about the same time; the Chemung reached a height at this place of 18 feet above low water; at Elmira the river began to rise rapidly about 9 a. m. of June 1 and was at its highest at about 7 p. m.

According to Francis Collingwood,[1] who investigated the Chemung River flood of 1889, for the city of Elmira, the foregoing data indicate that in a flood coming more from the south than from the west the highest water in Chemung River may be looked for about twelve hours after the highest water has passed the Tioga, and at a somewhat later period when the water comes more from the Canisteo and Tioga rivers. Mr. Collingwood concludes that the flood flow at Elmira will usually be not less than fourteen hours after a heavy rain in the head waters of these streams.

The surveys made by Mr. Collingwood, in which a considerable number of flood elevations were fixed and plotted, show that the discharge of Chemung River at its maximum was about 138,000 cubic feet per second, and the mean velocity 12.72 feet per second. A maximum of 138,000 cubic feet per second, gives 67.1 cubic feet per second per square mile. By the way of comparison, it may be noted that a maximum has been recorded on the Croton watershed of 74.87 cubic feet per second per square mile; also that Genesee River at Mount Morris gave in the flood of 1894 the maximum discharge of 32,000 cubic feet per second, or 48.6 cubic feet per second per square mile.

The Chemung flood of 1889 did considerable damage both at Corning and at Elmira, and the investigations of Mr. Collingwood were with reference to plans for protecting the latter city from devastation by future floods. Several plans were proposed, all including the rectification, clearing, and lowering of the river through the city with such dikes at the side as might be necessary for special protection at exposed points. The estimated cost of these various projects varied from $336,000 to $700,000. So far as known, nothing in the way of constructing the work at Elmira has yet been done.

The city of Corning, which is situated on the banks of Chemung River a few miles above Elmira, was also greatly damaged by the flood of June, 1889. In consequence, it was determined to construct protective works, and an act of the legislature was accordingly passed in 1892, creating a board of river commissioners, with authority to issue bonds for this purpose, under which enactment and amendments thereto bonds to the amount of $150,000 have been issued. The work began in June, 1896, and is now about completed. The plan adopted is to construct earthen dikes to confine the river at all points where it is subject to overflow. The total length of the dikes is about 25,800 feet, or 4.9 miles, and they vary in height from 4 to 19 feet. The river dikes were generally 8 feet wide on top, with a slope of from 3 to 1 (3 horizontal to 1 vertical) on the river side, and a slope of 2 to 1 on the land side.

Whatever the purpose for which an inland stream is to be utilized,

[1] Report on the Prevention of Floods at Elmira.

the first question asked by an experienced engineer is with regard to the minimum flow. If for power development, the minimum flow will determine the amount of power which can be insured on a given head; if for the water supply of a town, the minimum flow will indicate at once the number of people which may be supplied without storage. From every point of view, therefore, a knowledge of the minimum flow is a matter of the first importance. Below are given the minimum flows of the inland streams of the State of New York so far as information is at hand.

LOW-WATER FLOW OF OATKA CREEK.

The drainage area of this stream above the point of measurement is 27.5 square miles. The mean flow for the month of August, 1891, was 6 cubic feet per second; for September, 5.83 cubic feet per second; for October, 5.8 cubic feet per second. Expressed in cubic feet per second per square mile, the foregoing results are 0.218 cubic foot for August, 0.212 cubic foot for September, and 0.211 cubic foot for October. Expressed in inches on the watershed, the run-off of this stream for August to October, 1891, was from 0.24 to 0.25 inch per month. For several days during the months of August to October, 1891, the flow of Oatka Creek was down to about 4.2 cubic feet per second, or to about 0.151 cubic foot per square mile per second. On September 26, 1891, the recorded mean flow for the the day was 3.77 cubic feet per second, or 0.137 cubic foot per square mile per second.

As a general proposition, statements of minimum flows of streams ought not to be based on the record of single days, especially on streams where there are mill ponds above the point of measurement, because such accidental circumstances as the holding back of the water may vitiate the result; from this point of view an average extending over as long a period as possible should be taken.

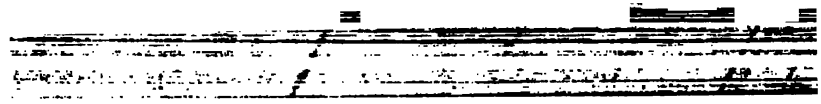
The measurements of Oatka Creek from August to October, 1891, illustrate well the nearly universal tendency of streams to run either at approximately a fixed rate or to decrease only very slowly after the tributary ground water has become well drawn down. For several days at a time the records show only slight variations.

LOW-WATER FLOW OF GENESEE RIVER.

The drainage area above Mount Morris, the first point of measurement, is 1,070 square miles; above Rochester, the second point, 2,365

A. ERIE CANAL AQUEDUCT AND SOUTH SIDE OF MAIN STREET BRIDGE, ROCHESTER

B. GREAT FLOOD OF 1865 AT ROCHESTER, SHOWING LUMBER LODGED AGAINST AQUEDUCT BRIDGE



ıre miles. The following table gives the mean monthly flows at nt Morris and Rochester for several low months of the year 1895:

Mean monthly flow of Genesee River at Mount Morris and Rochester.

Month.	Mount Morris.			Rochester.	
	Mean flow (cubic feet per second).	Cubic feet per second per square mile.	Inches on the water shed.	Mean flow (cubic feet per second.)	Cubic feet per second per square mile.
May	174	0.163	0.19	385	0.380
June	128	0.119	0.13	283	0.226
July	105	0.099	0.11	232	0.165
August	115	0.108	0.12	254	0.169
September	100	0.093	0.10	221	0.106
October	104	0.097	0.11	230	0.093

ımparing the foregoing figures for Mount Morris with those for ıester for the month of October, 1895, it is seen that the propor- of run-off at Rochester was somewhat less for that month than ount Morris, although for the previous months it appears to have larger. The explanation of this is that there are between ıester, Mount Morris, and Dansville extensive flats aggregating 60 to 80 square miles. The temporary ground-water storage of flats acts to sustain a somewhat more equable flow at Rochester at Mount Morris, above which point there are proportionately ı smaller areas of flats.

ıe foregoing minimum flows of Genesee River show conclusively in its present condition it is not a good mill stream. The great ıtions in run-off are conclusive on this point. The figures show the run-off of the stream may be exceedingly slack during the ner and fall months.

the summer of 1846 Daniel Marsh made a series of measure- ts in order to determine the low-water flow of that year. As the lt of 9 measurements made at various times in July and August ılaced the minimum flow at Rochester in 1846 at 412 cubic feet ıecond.

we examine the meteorological records of western New York for years 1844 to 1846, we find that the period covered was one of low fall. For instance, at Rochester the rainfall for the storage period ıe year 1846 (from December, 1845, to May, 1846, inclusive) was 11.57 inches; the rainfall of the growing period, 11.30 inches; he replenishing period, 13.16 inches; the total for the water year being 36.03 inches. For 1845 the total was 34.66 inches. For 1844 storage period rainfall was 10.52 inches; growing period, 8.23 es; replenishing period, 7.68 inches; total for the year, 26.43 inches.

At Middlebury Academy, Wyoming County, in the drainage area of Oatka Creek, the rainfall for the water year 1845 was, for the storage period, 12.59 inches; growing period, 4.82 inches; replenishing period, 8.60 inches; total for the year, 26.01 inches. The record for the year 1846 at Middlebury is not given. It is clear, therefore, so far as we have any definite meteorological record, that the measurements made by Mr. Marsh in 1846 were at a time of very low water.

The foregoing statements indicate that apparently the minimum summer flow of Genesee River has decreased from 462 cubic feet per second in 1846 to about 220 cubic feet per second in 1895. As to the reason for this decrease it is believed that the extensive deforestation of the drainage area which has taken place since 1846 offers full explanation. In 1846 the upper Genesee drainage area was still very largely in forest. Probably of the entire area above Rochester the virgin forest was from 65 to 70 per cent of the whole. We have, therefore, apparently a marked case where the deforestation of a large area has materially reduced the minimum run-off.

LOW-WATER FLOW OF HEMLOCK LAKE.

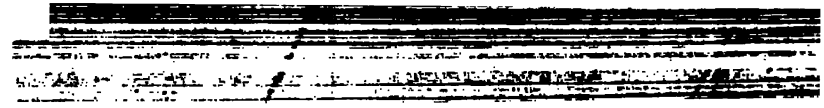
According to a report made by Henry Tracy, the minimum flow of Hemlock Lake (drainage area 43 square miles) is 5 cubic feet per second, or 0.116 cubic foot per square mile per second.[1]

The table on pages 76 and 77 gives, as previously stated, the quantity of water passing out of Hemlock Lake for the period covered and without reference to the natural flow. In order to obtain the approximate natural flow for the year we must take into account the mean elevations of lake surface. Thus, for the water year 1880 the mean elevation of the first month, December, was —1.67, while for the last month, November, it was —1.24. The difference (0.43 foot) represents the gain in depth of storage for the year. Computing for the value of this storage in inches on the drainage basin, we have 0.2[illegible] inch, which, added to the quantity of water passing out of the lake (3.07 inches), gives as the approximate total run-off for the year 3.3[illegible] inches. Since 1880 was a very dry year, we may compute the flow for the entire water year to be 10.3 cubic feet per second, which again amounts to 0.24 cubic foot per square mile per second. So far as known this is the lowest annual run-off thus far measured in the State of New York.

For the five-year period included in this table, the total rainfall and run-off are as follows:

[1] Report on the cost and policy of constructing reservoirs of Conesus, Hemlock, Honeoye, and Canadice lakes. Senate Document No. 40, 1850.

UPPER FALLS OF GENESEE RIVER AT PORTAGE



Rainfall and run-off of Hemlock Lake.

Water year.	Rainfall.	Run-off.
	Inches.	*Inches.*
1880	21.99	3.07
1881	24.27	8.38
1882	25.46	14.51
1883	32.24	9.29
1884	26.74	12.57
Add for rise in level	.....	0.40
Total	131.70	48.22

'or the five-year period the total run-off was therefore only 36.6 · cent of the rainfall. In 1880 the run-off was only 15.2 per cent of ı rainfall.

LOW-WATER FLOW OF MORRIS RUN.

The result of a measurement of Morris Run, a tributary of Oatka eek, the source of a part of the water supply of the village of arsaw, Wyoming County, made from July 4 to December 26, 94, is shown by the accompanying table. The measured drainage ea is 156 acres, but it may, by reason of the peculiar topography, be newhat greater than this. The water issues along the thread of e short valley in the form of springs. The measurement was made a thin-edged notched weir at a point just below the lowest spring. ı may be observed, the flow varied greatly at different times, the nimum being 77,630 gallons per day or 7.2 cubic feet per minute, in tober. On July 8 the discharge was 238,580 gallons, or 22.1 cubic et per minute for twenty-four hours. There is a popular impres-n that springs do not vary their flow at different seasons. The asurements of Morris Run are valuable, therefore, as illustrating ıt even a spring-fed stream will gradually decrease during a dry ison.

ly mean discharge in cubic feet per minute of Morris Run near Warsaw, New York, in 1894.

Day.	July.	August.	September.	October.	November.	December.
1	.....	16.0	10.4	.....	10.9	.....
2	.....	17.3	.....	.....	.....	.....
3	.....	16.7	8.3	.....	.....	10.0
4	19.4	18.3	8.0	7.8	.....	.....
5	17.8	.....	7.8	8.2	.....	.....
6	19.5	16.2	.....	.....	9.4	.....
7	20.9	14.9	.....	6.7	.....	.....

Daily mean discharge in cubic feet per minute of Morris Run near Warsaw, York, in 1894—Continued.

Day.	July.	August.	September.	October.	November.	Decem
8	22.1					
9	21.2					
10	21.3					
11	20.8			9.4		21
12	20.6			7.8		
13	19.7					16
14	20.1					
15	19.7			8.8		14
16	19.3		8.8		8.2	
17	18.7		8.2	7.8		
18	18.3		19.0	7.8		13
19	17.5			7.2	8.2	
20	17.9					13
21	21.2		11.1	7.2	8.2	
22	20.6		8.8		8.8	
23	20.0			7.2		
24	20.4		8.8	53.8	8.2	13
25	18.7					
26	17.8			8.8	8.2	13
27	17.3			7.8		
28	17.1		8.8			
29	17.8		8.8	7.2		
30	16.3			7.2		
31	16.5	11.7		19.0		

LOW-WATER FLOW OF WEST BRANCH OF CANADAWAY CREEK.

In the summer of 1883 measurements were made of the W Branch of Canadaway Creek in Chautauqua County, from July 1 September 2 of that year. This stream, which is the source of water supply of the village of Fredonia, has a drainage area at the point of measurement of 4.3 square miles. The valley is deep for a distance of 3 miles from the measuring point to its extr headwaters. Small springs issue frequently throughout the va On July 18, 1883, the stream was flowing at the rate of 541,620 lons in 24 hours, or 50.2 cubic feet per second, and very gradu decreased to 270,000 gallons, or 25 cubic feet per minute, on Jul Rains between July 22 and July 29 brought the stream up to a charge of 1,319,000 gallons per day, or 122.1 cubic feet per mi on the latter date. The flow then gradually decreased during month of August until, on August 26, it was only 216,000 gallon

y, or 20 cubic feet per minute, which was the lowest point reached ring the summer of 1883.

This stream can not be considered a good water yielder. A mean scharge of 216,000 gallons in twenty-four hours from a drainage ea of 4.3 square miles represents a yield of 0.334 cubic foot per cond, or, what is the same thing, 0.078 cubic foot per square mile r second. It is apparent, therefore, that even a spring-fed stream th a deep valley in Chautauqua County may at times furnish a ry small outflow, though it should not be overlooked that the flow 0.078 cubic foot per square mile per second was the extreme minium for one day only. The relations of this extreme minimum the daily flows during the period covered by the measurements ay be easily gathered from an inspection of the table. The gradual lling in water yield from August 1 to 26 is the most interesting fact vealed by these measurements.

The following was the rainfall at the point of measurements during e month of August, 1883:

	Inches.
igust 3	0.04
igust 13	0.10
igust 20	0.05
igust 23	0.05
igust 28	1.98

ily mean discharge in cubic feet per minute of West Branch of Canadaway Creek, near Fredonia, New York.

[Drainage area, 4.3 square miles.]

Day.	July.	August.	September.	Day.	July.	August.	September.
1		56.5	47.6	17		31.7	
2		50.3	45.0	18	50.2	28.4	
3		46.2		19	49.3	29.3	
4		45.4		20	45.7	31.7	
5		39.8		21	44.0	33.2	
6		36.9		22	25.0	22.9	
7		36.0		23		25.6	
8		34.1		24			
9		34.1		25		22.1	
10		33.2		26		20.0	
11		31.7		27		21.3	
12		30.2		28		105.1	
13		30.2		29	122.1	321.9	
14		32.4		30		101.7	
15		27.0		31	62.4	60.0	
16		27.9					

LOW-WATER FLOW OF SKANEATELES LAKE.

So far as can be learned, no definite statements of the minimum flow from this drainage basin, having an area of 73 square miles, have ever been made. Before the taking of the waters of this lake for the supply of the city of Syracuse the supply was ample for the canal, and close estimates were not made. For the run-off of a water year we find by the table on page 78 that 1897 was the lowest thus far measured, the total of that year being 15.61 inches on the watershed. The indications of the table, so far as they go, are that the Skaneateles area is a good water yielder. Nevertheless, it is improbable that 1897 was a year of minimum flow.

LOW-WATER FLOW OF OSWEGO RIVER.

There are no records of any long-continued measurements of the discharge of Oswego River, whose drainage area at the mouth is 5,613 square miles. The minimum flow of this stream has been the subject of judicial inquiry. In August, 1875, in the case of Michael J. Cummings against owners and lessees of the water of the Varick Canal at Oswego, it was decreed:

> (1) That the average flow of water from the Oswego River into the Varick Canal in low water in the summer months is about 45,000 to 50,000 cubic feet per minute; (2) that in extreme low water in the summer, and which usually occurs in the month of July or August, it is about 35,000 cubic feet per minute; (3) that the average flow of the whole three summer months is about 75,000 cubic feet per minute.

Varick Canal is entitled to receive one-half the total flow of the river, less the amount of water required for navigation purposes. Hence the average summer flow, according to the decree, is from 90,000 to 100,000 cubic feet per minute (1,500 to 1,670 cubic feet per second). The extreme low-water flow is placed at 70,000 cubic feet per minute for the whole flow of the river, or at 1,170 cubic feet per second, while the average flow of the whole three summer months is given at about 150,000 cubic feet per minute, or 2,500 cubic feet per second. From the foregoing figures we deduce an extreme minimum of perhaps 0.23 of a cubic foot per square mile per second, with an average of low water in the summer months of about 0.30 to 0.33 of a cubic foot per square mile per second.

LOW-WATER FLOW OF BLACK RIVER.

The drainage area of this stream at Watertown is 1,820 square miles. There is very little definite information as to either the maximum or the minimum flow. Aside from a few measurements made by engineers in the employ of the State at the time of construction of Black River Canal and a few made by Frank A. Hines in 1875, there do not appear to be any measurements of flow. As stated in the Report on Water Power of the United States, Tenth Census, probably the minimum flow

HIGH FALLS OF MOHAWK RIVER AT TIME OF LOW WATER.

an ordinarily dry season may be taken at from 1,000 to 1,100 cubic t per second for 24 hours. By reasoning from the data of Hudson ver, it may be assumed that in very dry years the minimum flow will less than this. Taking into account the large surface storage on numerous lakes at the head waters, it is doubtful if Black River its natural state will, while present forestry conditions are mainned, ever go below about 0.4 of a cubic foot per square mile per ond, although it is claimed to have been less than this in 1849. As shown in detail in another place, there are a large number of reserirs upon the upper waters of this stream, which, if properly operated, y be expected to keep the low-water flow at a considerably higher ure than 0.4 of a cubic foot per square mile per second, that figure lating to the natural flow of the stream only.

LOW-WATER FLOW OF MOHAWK RIVER.

The drainage area of this stream is 3,400 square miles. According David H. Van Auken, the engineer of The Cohoes Company, at hoes, the minimum flow of Mohawk River does not exceed 800 cubic et per second, or 0.235 cubic foot per square mile per second. Since nsiderable water is taken from Mohawk River for the supply of ie Canal, probably Mr. Van Auken's statement relates to the amount lized for water power at Cohoes, and, in the absence of definite ures as to the amount abstracted for the canal, must be taken as newhat general. There is, however, a well-defined feeling that the nimum flow of the Mohawk has been gradually decreasing during last twenty or twenty-five years, due probably to decreasing orestation of the drainage area.

LOW-WATER FLOW OF HUDSON RIVER.

The drainage area above Mechanicville is 4,500 square miles. asurements have been made at Mechanicville since October, 1887, record of which to November, 1896, is presented in the tables page 82. The natural flow of this stream is somewhat obscured the presence of a considerable number of lumbermen's reservoirs its head waters, the total storage of which aggregates about 00,000,000 cubic feet. The month of minimum run-off for the ole period covered by the measurements was July, 1888, the mean the month being 1,537 cubic feet per second, or 0.39 inch on the tershed. For short periods the mean flow has been less than this. us, from August 14 to 19, 1890, the mean flow was 1,080 cubic feet second; also, from October 2 to 6, 1891, inclusive, the mean flow given at 1,080 cubic feet per second. For 4,500 square miles ainage area this gives 0.24 cubic foot per second. Taking the ersion for the supply of Champlain Canal into account, we have out 0.29 cubic foot per square mile per second as the actually served *minimum flow.*

The figures show, moreover, that the minimum of 0.29 of a cubic foot per second has occurred for only two periods, one of six days and the other of five days, a total of eleven days for the whole period covered by the measurements. For July, 1888, the mean flow, including the diversion which was then occurring for the supply of Champlain Canal, may be taken at 0.37 of a cubic foot per square mile per second. For October, 1891, the mean flow for the whole month was 1,472 cubic feet per second, or, including the diversion to the Champlain Canal, 0.36 of a cubic foot per square mile per second. In July, 1890, the mean flow for the month was 1,950 cubic feet per second, and in several other months, as July, 1893, July, 1895, and September and October, 1895, the mean monthly flow varied from about 2,600 to 2,700 cubic feet per second. Hence we may say that for any business where it is not absolutely indispensable to have permanent power, water power on Hudson River may be developed up to the limit of about 0.4 of a cubic foot per square mile per second, with a prospect of not being interrupted on account of low water more than a few days in each year. For electric power, however, or any application of water power requiring a permanent power every day in the year, the development ought not to be based, under present conditions, on more than about 0.24 to 0.25 of a cubic foot per square mile per second, these latter figures relating especially to that portion of the river from which water is diverted for the supply of Champlain Canal. At points above the Glens Falls feeder the indications of the available data are that permanent power developments may be made up to 0.3 of a cubic foot per square mile per second. As is shown in the section on the water power of Hudson River, nearly all of the plants on that stream are developed far beyond these figures.

LOW-WATER FLOW OF CROTON RIVER.

The drainage area above the point of measurement is 338 square miles. The minimum flow of the main Croton River above the point of measurement, as given by J. J. R. Croes, is 0.178 of a cubic foot per square mile per second. The minimum flow of West Branch of Croton River, with a drainage area of 20.4 square miles, is given at 0.02 cubic foot per square mile per second. The lowest mean monthly flow in the period from 1870 to 1896, covered by the table on pages 83 to 85, is for the month of September, 1870, in which month the average daily run-off was 69,401,200 gallons or 9,265,800 cubic feet in 24 hours. These figures give 7.5 cubic feet per second and 0.318 of a cubic foot per square mile per second.

SUMMARY OF KNOWLEDGE OF LOW-WATER FLOW.

Summarizing the present knowledge of the minimum flow of streams in New York, we may say that in western New York for streams like Genesee River, issuing from regions of heavy, compact soil, mostly

deforested, the minimum flows are likely to run as low as 0.1 of a cubic foot per square mile per second, and even less. Spring-fed streams in that region, and those with considerable lake-surface pondage, may be expected to be somewhat greater than this. For the central part of the State the information is too limited to permit of making other than general statements. It is probable, however, that Oswego River, the main stream of the central region, has by reason of its lake pondage and swamp area a minimum flow not much less than 0.3 of a cubic foot per square mile per second. The Mohawk and the upper Hudson may also be placed, while their present condition of forestation is maintained, at a minimum of about 0.3 of a cubic foot per square mile per second. This figure also appears to apply to the Croton drainage area, where there are considerable sand areas, which compensate for the limited forestation. The streams of Long Island issuing from sand plains will give larger yields, the measurements showing a run-off of 0.58 cubic foot per second per square mile drained. The streams of the northern part of the State, which issue from denser forests than the others, may be expected to give minimum yields somewhat in excess of 0.3 of a cubic foot per square mile per second. Little is known as to the yield of streams tributary to the Allegheny, Susquehanna, and Delaware rivers aside from the measurements of Eaton and Madison brooks by Mr. Jervis, in 1835. Apparently no measurements of any other of these streams have been made. It is probable, however, that many of them are not specially different in water-yielding capacity from the Neshaminy, Tohickon, and Perkiomen creeks in Pennsylvania.[1]

In view of the vast importance of a detailed knowledge of stream flow in the State of New York, on account not only of the canal interests of the last seventy-five years, but also on account of the great possibilities of water-power developments, it is a matter of surprise that more extended measurements of the inland streams have not been made.

[1] For details of the measurements of the Neshaminy, Tohickon, and Perkiomen creeks, see a paper, Observations on rainfall and stream flow in eastern Pennsylvania, by John E. Codman. Proc. Eng. Club of Philadelphia, Vol. XIV (July to September, 1897).

[For index, see Part II of this report—Water-Supply Paper No. 25.]

DI

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

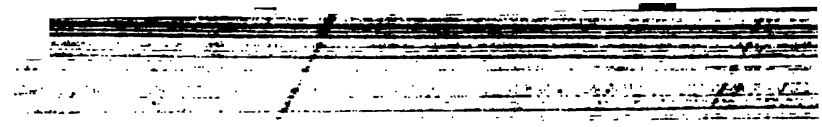
IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 25

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899



J. C. Branner

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

WATER RESOURCES

OF THE

TATE OF NEW YORK

PART II

BY

GEORGE W. RAFTER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1899

CONTENTS.

	Page.
tter of transmittal	107
ater-storage projects	109
Genesee River storage reservoir	109
Preliminary investigations	110
Interests to be served	111
Investigation of the flood of 1865	112
Mount Morris sites	113
Portage site	114
Comparison of Mount Morris and Portage sites	122
Summary	124
Water storage on Hudson River	125
Early surveys	125
Recent investigations	127
Reservoir sites on Sacundaga, main Hudson, and Schroon rivers	128
Effect of proposed storage on river flow	132
Summary	134
evelopment of water powers	135
Power development at Niagara Falls	135
Niagara Falls Hydraulic Power and Manufacturing Company	136
Niagara Falls Power Company	138
Power plant at Massena, on St. Lawrence River	143
land waterways	144
Trade and commerce of Hudson River	144
State canals	145
Early history	145
Growth and decline of canal transportation	151
Cost and revenues of the New York State canal system	154
Improvement of Erie Canal	155
Description of the canals now in operation, and their water supply	157
Eastern division of Erie Canal	158
Water supply of the eastern division	158
Middle division	161
Reservoirs of the middle division	164
Western division	165
Ship-canal projects and water supply	166
Loss of water from artificial channels	173
se and value of water power	178
Water power of Erie Canal	178
Power at Black Rock	178
Power at Lockport	179
Power at Medina	182
Selling price of water power	184
State ownership of inland waters	186
Future use of water power in New York	188
bstructive effect of frazil or anchor ice	190
Water yield of sand areas of Long Island	191
dex	199

ILLUSTRATIONS.

Page.

PLATE I. Upper fall of Genesee River at Rochester, New York, at time of flood 112

II. East side of Genesee Gorge at site of proposed Portage dam 114

III. *A*, Lumberman's dam on Cedar River; *B*, Lumberman's dam on Indian River 120

IV. *A*, The George West Paper Mill on Hudson River at Hadley, New York; *B*, Hudson River Pulp and Paper Company's mills at Palmer Falls on Hudson River, New York 122

V. *A*, Glens Falls Paper Mill at Fort Edward, New York; *B*, Dam of the Hudson River Power Transmission Company during construction 124

VI. Power house of the Niagara Falls Hydraulic Power and Manufacturing Company 136

VII. Penstock of the Niagara Falls Hydraulic Power and Manufacturing Company 138

VIII. *A*, Power house of Niagara Falls Power Company; *B*, Outlet of tunnel of Niagara Falls Power Company 140

IX. *A*, Erie Canal at Buffalo, New York; *B*, Black Rock guard lock on Erie Canal 148

X. Effect of decrease of business on Erie Canal 152

XI. Erie Canal at Little Falls, New York 156

XII. Erie Canal at Syracuse, New York 160

FIG. 1. Map of drainage area of Hudson River above Glens Falls 126

2. General plan of development of the Niagara Falls Hydraulic Power and Manufacturing Company 137

3. Map of Niagara Falls and vicinity showing location of the great tunnel 139

LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY.
Washington, November 26, 1898.

IR: I have the honor to transmit herewith a manuscript on the ater Resources of the State of New York, prepared by Mr George . Rafter, and to recommend that it be published as one of the series papers on Water Supply and Irrigation.

This manuscript is a continuation of the material printed in Water ipply Paper No. 24, which related mainly to the physical conditions river systems of the State of New York, and particularly to the vailable water supply, the floods, and the low water of some of the pical streams. In the present paper a discussion is given of water orage projects and of the development of water power and water ays. The paper as originally presented was intended for publication a whole in one of the annual reports. This was found impracticable, and to secure publication at an early date it has been necessary subdivide the manuscript to conform with the limitations imposed garding the size of papers in this series. In spite of this disadvantage it is believed that the information contained in these two papers (Nos. 24 and 25) will be of value not only to the people of New York, ut to those of other States.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

ATER RESOURCES OF THE STATE OF NEW YORK, PART II.

By GEORGE W. RAFTER.

WATER-STORAGE PROJECTS.

Among the projects for storing water for power and other purposes e two of special importance with which the author has been con- rned; the first of these is on Genesee River and the second on udson River. The following statements are for the most part con- nsed from more detailed reports in the Annual Report of the State gineer and Surveyor for 1896.

GENESEE RIVER STORAGE RESERVOIR.

A general description of this river has been given in Part I (Water- pply Paper No. 24), on page 25; its discharge measurements have n discussed on page 70, and reference has been made on page 90 the low-water flow, indicating that during the summer the avail- le supply is small. In spite of this fact, development of water wer has proceeded rapidly. As shown by the reports on the Water wer of the United States in the Tenth Census (1880), the total ter power on Genesee River from Rochester to Portage in 1882 was 82 net horsepower. An examination of the amount in use on the ne reach of river in 1896 showed that the total, based on manu- turers' rating of wheels, was 19,178 net horsepower, or based on e manufacturers' statements of the quantity of water required to erate the wheels, and allowing 75 per cent efficiency of the water, e total power developed by the wheels in place in 1896 is found to 17,248 net horsepower, or about three times that in 1882. In com- rison with the figures it should be noted that for several months ring the summer and fall of 1895 the total power did not exceed 000 horsepower. The same condition has existed during the dry riod of a number of years previous, but not so seriously as in the ll of 1895.

PRELIMINARY INVESTIGATIONS.

The increased demand for power, as well as the serious summer droughts, led to the formulation of a project for constructing a storage reservoir at some point on the head waters of Genesee River for assisting the summer flow. The first project included the development of the basin of Honeoye Lake to its full capacity, surveys having been made for that purpose in 1887 and 1888. It appeared, however, that the yield of this drainage basin, which is only about 43.5 square miles, was hardly adequate for the results desired, the estimate showing that even when developed to its full capacity it could not be depended on to furnish, in a dry year, more than 75 cubic feet per second, while the exigencies of the case demanded at least several hundred cubic feet per second. The project of building a large storage reservoir on upper Genesee River was then formulated by the Rochester Chamber of Commerce.

In the meantime a number of breaks on the long level of the Erie Canal, which extends from the foot of the locks at Lockport to the eastern part of the city of Rochester, a distance of about 62.5 miles, had emphasized the importance of the State's providing additional water for feeding the canal east of Rochester. For this purpose the construction of a large storage reservoir was advocated by the Rochester Chamber of Commerce as a State work, with the result that under a resolution of the senate dated March 21, 1889, the State engineer and surveyor was directed to make a general investigation in regard to the possibility of storing water on the upper Genesee. The report made under the authority of this resolution appears in the Annual Report of the State Engineer and Surveyor for the year 1890. In 1892, under authority of a concurrent resolution dated March 15 of that year, Governor Flower appointed a commission consisting of Evan Thomas, Judge Charles McLouth, and John Bogart to investigate and report on the whole question of storage on the upper Genesee. This commission examined the site of the proposed reservoir and reported that it was entirely feasible to construct a large reservoir on the upper Genesee River, the site especially considered by the commission being in the Genesee canyon or gorge, a short distance above Mount Morris, already described in Part I.

As the result of the recommendations of this commission, the sum of $10,000 was appropriated at the legislative session of 1893 for the purpose of studying in detail the several proposed sites for dams in the canyon of Genesee River, above Mount Morris. At that time the work was placed in charge of the author, and has since remained in his hands.[1]

At the legislative session of 1894 a bill to construct a dam in the canyon a short distance above Mount Morris passed the senate, but

[1] *The result of* the studies in 1893 may be found in the Annual Reports of the State Engineer *and Surveyor for* the fiscal years ending September 30, 1893 and 1894.

d in the assembly. At the session of 1895 a similar bill passed the senate and assembly, but was vetoed by Governor Morton, ly on the ground that the bill as passed made no provision for owners of the water power and other interested parties bearing portion of the expense. In his veto Governor Morton expressed belief that if the State should determine to build a dam on see River some provision should be made by which the city of ester—and possibly other localities interested in the work— t contribute to the expense of construction. Governor Morton pointed out that if the proposed canal enlargement be approved e people public sentiment might justify the construction of a ge dam on Genesee River for canal purposes. On the other l, if the proposition to deepen the canal should not be approved question would still remain whether such a dam might not be able for the purpose of regulating the river and increasing the r power thereon.

order to complete the preliminary investigations relative to the osed Genesee storage, Governor Morton, in 1896, approved an tional appropriation, which was expended during the summer of year in completing the additional surveys. To the present time State has expended on preliminary investigation of the Genesee ge project the following amounts: In 1890, $3,000; in 1892, $7,000; 93, $10,000; in 1896, $10,000; in all, $30,000. As a result of this nditure complete plans and specifications have been prepared as n in the Annual Report of the State Engineer and Surveyor for 1

INTERESTS TO BE SERVED.

e following are the interests to be served by the construction of extensive storage works on Genesee River:

) The flow of the river would be regulated, thus effectually pre- ing in the future the devastating floods which occurred in 1815, , 1857, 1865, 1889, 1893, 1894, and 1896. The floods in the years just

way of presenting a full list of the work on the Genesee storage, reference may be made special report of John Bogart, State engineer and surveyor, in Appendix F of the Annual t of the State Engineer and Surveyor for the fiscal year ending September 30, 1890. The s of Messrs. Bailey and Kibbe, assistant engineers to Mr. Bogart, are covered by the same nce. The report of Martin Schenck, State engineer and surveyor, may be found at page he Annual Report of the State Engineer and Surveyor for the fiscal year ending Septem- , 1893. The report of E. Sweet, ex-State engineer and surveyor, as consulting engineer, e found in Appendix H of the Annual Report of the State Engineer and Surveyor for the year ending September 30, 1893. The report of the commissioners appointed in 1892 by nor Flower may be found in Senate Doc. No. 23, 1893. The first report of the writer may nd in Appendix G of the Annual Report of the State Engineer and Surveyor for the year ending September 30, 1893. The second report may be found in Appendix E of the l Report of the State Engineer and Surveyor for the fiscal year ending September 30, 1894. ork done in 1896 is described at length in the Annual Report of the State Engineer and yor for the fiscal year ending September 30, 1896. See also a paper by the writer, The se River storage and its relations to the Erie Canal and the manufacturing interests of n New York, prepared for the Rochester Chamber of Commerce. This paper contains amount of historical information not given in the official reports. Governor Morton's ay be found in the Governor's State Papers for 1895.

enumerated were especially severe, but floods not so severe, yet doing considerable damage, have occurred in several of the intervening years. The most severe flood was that of 1865, which destroyed full $1,000,000 worth of property in the city of Rochester.

(2) Water would be supplied for the enlarged Erie Canal. According to Mr. Bogart's report of 1890, there should have been provided a storage on Genesee River of 1,500,000,000 cubic feet for the purpose of supplying Erie Canal as it existed at that date. Estimates made in 1896 show that for the purpose of fully supplying the enlarged canal, as now in process, there should be made a storage on Genesee River of 2,500,000,000 cubic feet.

(3) The agricultural production of the broad level area included in Genesee Valley between Rochester, Mount Morris, and Dansville, estimated at from 60 to 80 square miles, might be greatly increased by moderate irrigation if the flood contingency was removed and the proper irrigation channels were constructed.

(4) Considerable sanitary benefit to this section would result from the increased flow during the low-water period through the proposed regulation. The entire sewage at Rochester, a city of 160,000 inhabitants, now passes into Genesee River. The channel of this stream, between the foot of Lower Falls at Rochester and Lake Ontario, is so broad and deep that during the time of extreme low water in the summer and fall the current is scarcely perceptible. The sewage of the city therefore lodges in this section, producing a serious nuisance. The regulation of the river, by preventing floods, would also improve the sanitary condition of the broad upper valley, where the annual overflow has been shown to cause more or less sickness.

(5) The water power would be increased. Wheels are now set on Genesee River capable of producing, at the manufacturers' rating, 19,178 horsepower, while the low-water flow of the stream does not exceed about 6,000 horsepower. The regulation as finally proposed would produce much more power than this.

In summation of the preceding points it may be urged, in general, that in constructing the proposed Genesee storage dam, in addition to the private interests to be conserved, public service of an extended character would be performed.

INVESTIGATION OF THE FLOOD OF 1865.

The great flood of March, 1865, worked such destruction that the legislature, on May 1 of that year, passed an act appointing commissioners to ascertain the cause of the inundation of the city of Rochester, what obstructions, if any, had been placed in the river, and what measures, proceedings, and remedies were necessary to guard against the recurrence of such an inundation. The commissioners appointed under this act examined carefully the evidence as to the flood, and arrived at the conclusion that there were three principal causes:

UPPER FALL OF GENESEE RIVER AT ROCHESTER, NEW YORK, AT TIME OF FLOOD

1) The sudden melting of an immense body of snow which had umulated during the previous winter.

2) Obstructions caused by the bridge and embankment of what is w the New York, Lake Erie and Western Railway at Avon. The nings at this place are stated to have been adequate for ordinary ds, but entirely too small for the quantity flowing in March, 1865. nce, at the time of greatest flow, the water stood three feet higher the upper side of the embankment than on the lower side. The bankment finally gave way, thus allowing a large quantity of ided water to flow suddenly down the river, filling the channel at chester beyond its carrying capacity.

3) Obstruction of the channel of the river through the city of Roch- er in such manner as to cause overflows into the Erie and Genesee lley canals at that place. The commissioners also point out that construction of the Erie Canal aqueduct is such as materially to rease the obstruction at Rochester.

'rom the best available figures the maximum flow at Rochester in great flood of March, 1865, probably did not exceed about 40,000 ic feet per second. The danger limit is reached when the flow at t place approximates 33,000 cubic feet per second.

'he following are the openings of the several bridges, etc., span- g Erie Canal in the city of Rochester, as they exist at present: irt street bridge, 5,081 square feet; Main street, 3,367 square feet; drews street, 4,511 square feet; Central avenue, 4,450 square feet; e Canal aqueduct, 4,308 square feet.

s a chief cause of the 1865 flood the commissioners considered that ting off the forests and clearing lands were likely to lead to heavier ds from year to year. In view, therefore, of what seemed to the imissioners a constant source of danger, they arrived at the con- sion that a much larger waterway through the city of Rochester imperative. It may be here remarked that the waterway at chester is still substantially the same as in 1865; if anything, it been somewhat contracted by various constructions since that e.[1]

MOUNT MORRIS SITES.

Referring to Mr. Bogart's report of 1890, it is learned that the inves- tions of that year were general in their character. The work was ried on more particularly with reference to a location in Genesee er gorge, between Mount Morris and the foot of the Portage Falls. detailed surveys were made further than necessary to make a eral estimate of the cost of a dam 58 feet in height, which would re 1,500,000,000 cubic feet, the amount considered necessary for al purposes alone. Such a dam, Mr. Bogart estimates, could be cted for about $1,000,000.

---

or report of the commissioners appointed to investigate the causes of the inundation of the of Rochester in March, 1865, see Ass. Doc. No. 117 (1866).

The work performed under the direction of the author, in 1893, was of an entirely different character. The report of 1890 having indicated the Mount Morris Canyon as a desirable location, with a number of sites pointed out, of which general investigations had been made, it became desirable to investigate those sites in detail and to prepare close estimates of the cost of constructing dams at each. Detailed investigations were accordingly made of the three sites favorably reported upon in 1890, the results of which may be found in the Annual Reports of the State Engineer and Surveyor for 1893 and 1894, where estimates of the cost of the several dams are also given in detail. Referring to the estimates, it appears that at site No. 1, in Mount Morris Canyon, a dam raising the water surface 130 feet would cost, if built of concrete alone, $2,450,000, but if built with sandstone faces throughout, except for the spillway, where granite is provided, the estimated cost would become $2,590,000. A dam of the same height at site No. 2, if built throughout of concrete, would cost $2,600,000, but with sandstone faces and independent spillway the cost would be $2,720,500, or, with roadway, $2,785,000.

In regard to the total storage to be obtained in Mount Morris Canyon the following are the figures at sites Nos. 1 and 2, the two sites chiefly considered: At site No. 1 a dam 130 feet in height will store 7,700,000,000 cubic feet, and at site No. 2 a dam of the same height will store 7,040,000,000 cubic feet. Since no conclusion has been reached as to which of these sites to adopt, for the purposes of comparison a mean of 7,370,000,000 cubic feet has been taken as the approximate available storage, and the mean of $2,785,000 as the approximate total cost. On this basis the estimated cost of this storage becomes $377.88 per million cubic feet stored.

PORTAGE SITE.

As already stated, the investigations of the Genesee River storage project were finally completed in 1896. In that year detailed surveys were made of a new site known as Portage site (shown on Pl. II), the proposed dam to be located at Portage, about 1,400 feet above the Erie Railway bridge, at a point where the gorge presents extremely favorable conditions for the erection of a high dam. At this place solid rock exists immediately in the bed of the river, with only a couple of feet of water flowing over it, and also extends high up on the bluffs at either side, whereas at all of the sites in the gorge near Mount Morris the rock was only found at from 15 to 20 feet below the water surface and of such an open texture as to require cut-off trenches about 30 feet deep, or to a total depth of nearly 50 feet below the water. The proposed Portage dam is also 500 feet vertically above the previously mentioned dam site, thus rendering that additional number of feet available for power purposes—a fact which

EAST SIDE OF GENESEE GORGE AT SITE OF PROPOSED PORTAGE DAM

[redacted]

; a materially different aspect on the commercial side of the ;ee River storage project.

hort distance above the proposed Portage site the upper Genesee y broadens out to a width in places of from one to two miles, ıgh the general width of the valley does not exceed, for several in extent longitudinally, about one mile. It narrows in two or places to a less width than this. The valley is now a good agri-·al region, in a fair state of cultivation, and presents, on the , as favorable conditions for farming as any similar valley in tate. The Western New York and Pennsylvania Railway ; through the middle of the valley on the line of the abandoned ;ee Valley Canal. Along the line of this railway the villages rtageville, Rossburg, Wiscoy, and Fillmore are situated. The ·oir project includes the relaying of the railway above the flow n the west side of the valley, as well as the removal of the vil-named. The total area below the flow line is 12.4 square miles he entire area proposed to be taken for reservoir purposes, ling a strip 10 feet vertically above the flow line, is 13.7 square The project also includes the removal of several cemeteries, ıilding of highway bridges across the reservoir, and the con-ion of a roadway entirely around the same.

;hout having made a detailed canvass, it is estimated that the ıt population of the proposed Portage reservoir site, in the es and on the farms, is about 1,200. In reference to dispossess-is number of people of their homes for the purpose of creating e storage reservoir, it may be said that such a proceeding is not not uncommon in this State, but that the population to be ·ed in the case of the new Croton reservoir is far greater than at ortage reservoir. According to maps furnished by the Croton department, it appears that the new Croton reservoir includes king of either the whole or parts of something like three large es and nine or ten hamlets. The total population to be removed ;he submerged area of the new Croton reservoir is not given, but l inspection of maps of the proposed sites indicates that it must ·eral times larger than the number to be dispossessed at Portage. illages of Katonah, Purdy Station, and Croton Falls are much · than any of the villages in the Portage reservoir site. The line of the New York and Northern Railroad passes for several through the valley and requires relocating above the flow line, ,me as is proposed for the Western New York and Pennsylvania 'ay along the Portage reservoir. It appears, therefore, that the f New York is now doing under State laws everything in the ıf so-called radical change which it is proposed to do at Portage. th cases the sufficient reason for these changes may be found in ;tter meeting of public necessities.

The estimated cost of the proposed Portage reservoir, includ land damages, dam, reconstruction of railway, removal of cemeteri the cutting of all timber within the drainage areas, the construct of highway bridges, etc., is $2,600,000, the storage to be provided this expenditure amounting to 15,000,000,000 cubic feet; at this r the cost per million cubic feet stored is $173.33. The main charact istics of the proposed reservoir are shown by the following table, co densed from pages 708 and 709 of the report of the State Engineer a Surveyor of New York for 1896:

Capacity of proposed Portage reservoir.

Elevation of water surface above sea level.	Area of water surface.	Total volume of water in reservoir.	Inches on water-shed.
Feet.	*Sq. miles.*	*Cubic feet.*	
1,100.0	0.2320	101,400,000	0.044
1,105.0	0.5330	217,623,000	0.094
1,110.0	0.8340	333,900,000	0.143
1,115.0	1.1350	450,100,000	0.194
1,120.0	1.4357	566,300,000	0.244
1,125.0	2.0659	942,100,000	0.406
1,130.0	2.6692	1,318,000,000	0.567
1,135.0	3.3264	1,694,000,000	0.729
1,140.0	3.9566	2,070,000,000	0.891
1,145.0	4.5255	2,780,000,000	1.196
1,150.0	5.0944	3,490,000,000	1.502
1,155.0	5.6633	4,200,000,000	1.808
1,160.0	6.2322	4,910,000,000	2.113
1,165.0	6.8293	5,945,000,000	2.559
1,170.0	7.4264	6,980,000,000	3.004
1,172.0	7.6652	7,395,000,000	3.182
1,173.0	7.7846	7,602,000,000	3.271
1,175.0	8.0235	8,016,000,000	3.451
1,180.0	8.6206	9,051,000,000	3.896
1,185.0	9.4362	10,366,000,000	4.462
1,190.0	10.2518	11,681,500,000	5.016
1,195.0	11.3007	13,257,000,000	5,710
1,200.0	12,3495	15,000,000,000	6.458

Comparing the foregoing statements of cost with those made on tl preceding page with reference to the cost of the proposed reserv in Mount Morris Canyon, it appears that at Portage a storage 15,000,000,000 cubic feet can be made for somewhat less than the c

of 7,300,000,000 cubic feet at Mount Morris; or, as a general statement, we may say that a given expenditure at Portage produces double the storage that it will produce at Mount Morris. The Portage reservoir develops the full capacity of the drainage area for such a dry year as 1895. It is considered that this full development is necessary in order to obtain the most satisfactory results in river regulation.

As reasons in detail for preferring Portage site to that at Mount Morris, the following may be mentioned:

(1) The Portage site affords more water for a given expenditure.

(2) The Portage site is considered safer than the Mount Morris site. As shown in the Genesee Storage Reports of 1893–94, the shales at Mount Morris are open; and while it is, without doubt, possible to make a safe dam there, it would be at much greater cost than at Portage. In view of the large storage provided at either place, the dam must be absolutely safe, as its failure would work vast destruction.

(3) The material for the dam is nearly all on the ground at Portage, while at Mount Morris it needs to be brought from a distance.

(4) The Portage site affords greater water-power development. With the Genesee storage dam located at Mount Morris the total head at which the storage can be applied is 282 feet, while with a dam at Portage the total head on which the stored water may be applied is 2 feet.

(5) On account of great depth of foundation at Mount Morris, it would be necessary to expend over $1,000,000 before the dam could be brought to the level of the present water surface. The conditions are such that the floods of every spring would sweep over the work, obliterating all evidence that any money had been expended, which must necessarily continue for at least two or three years, until the foundations could be fully placed. At Portage, on the other hand, a good foundation is found near the water level. Indeed, the difference of cost of foundation is such that nearly the total expenditure at Mount Morris is for the dam, the flowage ground costing only $75,000, while at Portage the estimated cost of the dam is only $1,000,000, the balance of the expenditure there being for right of way, change of railway line, etc.

The proposed regulation of Genesee River has been computed on the basis of a minimum discharge of 300 cubic feet per second, in the case of a reservoir storing 7,500,000,000 cubic feet, and also on a basis of 457 cubic feet per second in the case of a reservoir storing 1,000,000,000 cubic feet. As to the reason for fixing upon these minimums, it may be remarked that in river regulation the outflow from the storage reservoir should be so arranged as to make the benefit to all parts of the stream equal. Especially is this proposition true when, as in the present case, there is water power distributed

throughout the whole extent of the stream below the storage point. Obviously the way to do this is to plan for an outflow proportional to the drainage area. In the present case we have a drainage area at Rochester of 2,365 square miles, and one of 1,000 square miles above Portage, or the area above Rochester is about 2⅓ times the area above Portage. The minimum regulated flow at Rochester may justly be made 2.365 times the assumed minimum flow at Portage.

Assuming 680 cubic feet per second as the flow below which the stream will never be allowed to fall at Rochester, we have for a reservoir storing 7,500,000,000 cubic feet a corresponding minimum outflow from the reservoir of 300 cubic feet per second, or for a storage of 15,000,000,000 cubic feet an outflow of 457 cubic feet per second, the latter figure being arrived at by assuming the maintenance of a minimum flow at Rochester of at least 1,080 cubic feet per second. The computations of the tables on pages 119 to 121 are carried out on this basis. The regulated flows for the month of May are greater than for the other months. They are also greatest during the months of canal navigation, the addition being made in order to provide for the quantity of water to be taken for the enlarged Erie Canal, which quantity has been fixed at 80 cubic feet per second for every month of the navigation season except May, and at 177 cubic feet per second for that month, the excess quantity for the month of May being required in order to provide for filling the canal at the beginning of the month.

The first table shows the effect on the flow of Genesee River from June, 1894, to November, 1896, inclusive, as influenced by the storage at Portage of 7,500,000,000 cubic feet of water, provided at least 300 cubic feet per second is allowed to flow continually at Portage, and at least 600 cubic feet per second is always flowing at Rochester in addition to the amount required for canal purposes. The figures given in the left-hand column show the proposed minimum flow at Rochester, this being the 600 second-feet above noted, together with 80 second-feet for the canal for the months from June to November, inclusive, and 177 second-feet for the month of May. The next two columns give the discharges at Rochester and Portage under natural conditions. The fourth column is the difference between these, or the quantity of water entering the river below Portage, and next to this is the minimum amount to be added at Portage in order to maintain the proposed minimum flow at Rochester of 600 cubic feet per second, not including the amount taken by the canal. The quantity available at Rochester for power purposes is shown in the next column, and is obtained by adding the flow at Rochester, less the flow at Portage, to the actual flow from Portage reservoir and the surplus flowing over the spillway at Portage reservoir, and then deducting the quantity taken by the canal.

Regulation of Genesee River by storage at Portage.

rage of 7,500,000,000 cubic feet at Portage and flow of at least 600 second-feet at Rochester.]

h.	Proposed minimum flow at Rochester.	Natural flow at Rochester.	Natural flow at Portage.	Flow at Rochester less flow at Portage.	Minimum amount to be added at Portage.	Available at Rochester.	Actual flow from Portage reservoir.	Surplus flowing over spillway.
.	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*	*Sec.-feet.*
.....	680	2,321	981	1,340	.....	2,205	300	645
.....	680	292	123	169	511	600	511	.....
.....	680	442	187	255	425	600	425	.....
ber ..	680	1,963	830	1,133	.....	1,353	300	.....
.....	680	899	380	519	161	739	300	.....
ber ..	680	1,729	731	998	.....	1,505	300	287
er ...	600	1,256	531	725	.....	1,241	300	216
',								
r	600	1,335	565	770	.....	1,330	300	260
y ...	600	495	209	286	314	600	314	.....
.....	600	3,985	1,684	2,301	.....	3,867	300	1,266
.....	600	4,257	1,800	2,457	.....	4,197	300	1,440
.....	777	385	163	222	555	600	555	.....
.....	680	283	120	163	517	600	517	.....
.....	680	232	98	134	546	600	546	.....
.....	680	254	108	146	534	600	534	.....
ber ..	680	221	93	128	552	600	552	.....
.....	680	230	97	133	547	600	547	.....
ber ..	680	993	420	573	107	793	300	.....
er ...	600	2,710	1,146	1,564	.....	1,864	300	.....
;.								
r	600	964	408	556	44	856	300	.....
y ...	600	2,005	848	1,157	.....	1,457	300	.....
.....	600	6,158	2,604	3,554	.....	4,999	300	1,145
.....	600	7,172	3,033	4,139	.....	7,155	300	2,716
.....	777	347	147	200	577	600	577	.....
.....	680	654	277	377	303	600	303	.....
.....	680	501	211	290	390	600	390	.....
.....	680	416	176	240	440	600	440	.....
ber ..	680	327	138	189	491	600	491	.....
'.....	680	3,667	1,556	2,111	.....	2,331	300	.....
ber ..	680	1,728	731	997	.....	1,486	300	269

The actual flow from Portage reservoir is taken to be the mini of 300 cubic feet per second ordinarily assumed, plus the an necessary to make up the deficiency in the quantity of water ent the river at points between Portage and Rochester. The last co that on the right-hand side, shows the surplus flowing over the way at Portage reservoir during the comparatively few months the natural flow has filled the reservoir, supplied all deman evaporation from the surface, and still is in excess.

The next table exhibits the condition of the reservoir from m to month under the above conditions. The figures are given i cubic contents but in equivalent depth in inches on the total t tary watershed of 1,000 square miles. The reservoir is assumed full at the beginning and end of June, 1894, the total storage o reservoir being equivalent to 3.23 inches in depth on the water The total waste from June 1, 1894, to December 1, 1896, equals, u the conditions of this table, 9.36 inches on the watershed.

Flow into and from Portage reservoir under the conditions assumed.

[In inches on watershed.]

Month.	Inflow to reservoir.	Outgo from reservoir.			Excess.	Deficiency.	In reservoir at end of month.	Wa
		Evaporation.	Amount to stream.	Total outgo.				
1894.								
June	1.10	0.03	0.33	0.36	.....	.....	3.72	
July	.14	.04	.59	.63	.....	0.49	2.76	....
August	.22	.04	.49	.53	.....	.31	2.45	....
September	.93	.03	.33	.36	0.57	.....	3.02	....
October	.44	.02	.35	.37	.07	.....	3.09	....
November	.82	.01	.33	.34	.48	.....	3.25	
December	.61	.01	.35	.36	.25	.....	3.25	
1895.								
January	.66	.01	.35	.36	.30	.....	3.25	
February	.22	.01	.33	.34	.....	.12	3.13	....
March	1.94	.01	.35	.36	1.58	.....	3.25	
April	2.01	.02	.33	.35	1.66	.....	3.25	
May	.19	.04	.64	.68	.....	.49	2.76	....
June	.13	.04	.58	.62	.....	.49	2.27	....
July	.11	.03	.63	.66	.....	.55	1.72	....
August	.12	.03	.61	.64	.....	.52	1.20	....
September	.10	.02	.62	.64	.....	.54	.66	....
October	.11	.01	.61	.62	.....	.51	.15	....
November	.47	.01	.33	.34	.13	.....	.28	....
December	1.32	.01	.35	.36	.96	.....	1.24	..

w into and from Portage reservoir under the conditions assumed—Cont'd.

[In inches on watershed.]

onth.	Inflow to reservoir.	Outgo from reservoir.			Excess.	Deficiency.	In reservoir at end of month.	Wasted.
		Evaporation.	Amount to stream.	Total outgo.				
896.								
ary	.47	.01	.35	.36	.11	.....	1.35	.....
uary ...	.91	.01	.32	.33	.58	.....	1.93	.....
h	3.00	.01	.35	.36	2.64	.....	3.25	1.32
l	3.38	.02	.33	.35	3.03	.....	3.25	3.03
.....	.17	.03	.67	.70	.....	.53	2.72	.....
.....	.39	.04	.34	.38	.01	.....	2.73	.....
.....	.24	.04	.45	.49	.....	.25	2.48	.....
ust	.20	.04	.51	.55	.....	.35	2.13	.....
mber ..	.16	.02	.57	.59	.....	.43	1.70	.....
ber	1.74	.02	.35	.37	1.37	.....	3.07	.....
mber ..	.82	.01	.33	.34	.48	.....	3.25	.30

milar tables might be given, showing the regulation of the river fected by the storage at Portage of 15,000,000,000 cubic feet of r for the same period, with at least 457 cubic feet per second ys flowing at Portage, and at least 1,000 cubic feet per second at nester, in addition to the amount required for the canal. The f difference is that during only three months of this period would e be any overflow through the spillway, the total waste equaling inches. This would be in June, 1894, 521 second-feet; in April, , 1,058 second-feet; in April, 1896, 350 second-feet. The amount ochester under the same conditions would be in no case below) second-feet.

may here be pointed out that had the enlarged canal been in ation in July, 1894, and taking the estimated quantity of 80 cubic of water per second from Genesee River, the amount of water g to the canal would have been 27.4 per cent of the total flow of river for that month; in August 18.1 per cent; in May, 1895, 46 cent; in June, 28.3 per cent; in July, 34.5 per cent; in August, per cent; in September, 36.2 per cent; and in October, 34.8 per . In May, 1896, the canal would have taken 51.3 per cent of the l flow of the river for that month; in June, 12.2 per cent; in July, per cent; in August, 19.2 per cent; and in September, 24.5 per . It appears, therefore, that the taking of 80 cubic feet per sec- from Genesee River for canal purposes is a very serious matter he water power of the stream and is unjustifiable, unless it be *rly shown that the* addition to the wealth of the State is greater

than if the water were used for supplying power. The actual damage resulting from taking at times 50 per cent of the unregulated flow of the stream is about as follows: As shown on a previous page, the minimum flow of the river is capable of producing 6,727 gross horsepower, or, what is the same thing, assuming 75 per cent efficiency, 5,046 net horsepower. One-half of the low-water power may therefore be taken at 2,523 net horsepower.

So long as the possibility exists of a draft upon the river equal to one-half of its minimum flow, this 2,523 net horsepower is practically rendered useless to its owners by reason of the uncertainty as to the exact time of the draft, or if not rendered useless, is far less valuable than if it were absolutely permanent power. In enforcing this view it may be pointed out that Rochester is a manufacturing town, made up chiefly of establishments using comparatively small quantities of power at each place, but that the power must still be continuous every day; that is to say, it must be absolutely permanent power. So long, therefore, as one-half the total minimum power of the stream is subject to stoppage during any month, the manufacturers will preferably use steam power, on account of its permanency, even at considerably greater expense. Bearing on this view, the fact may be pointed out that the use of soft coal in Rochester for steam purposes is stated as 500,000 tons a year, which, at an average price of $2.40 per ton, amounts to the sum of $1,200,000 annually. It may be considered settled, therefore, that water power is valuable at Rochester, and that anything which tends to reduce the permanent power 50 per cent is a very serious matter to the manufacturers of the city.

COMPARISON OF MOUNT MORRIS AND PORTAGE SITES.

As a final point in the discussion of Genesee River storage, comparison will be made between the Mount Morris project, storing 7,370,000,000 cubic feet, at a cost of $2,785,000, and the Portage project, storing 15,000,000,000 cubic feet, at an estimated cost of $2,600,000, for the purpose of determining the relative commercial advantages.

With the reservoir at Mount Morris storing 7,370,000,000 cubic feet there is 282 feet fall, on which 7,370,000,000 cubic feet, less the quantity required for the canal, may be applied for power purposes. As already explained, the constant outflow from the reservoir would never be less than 300 cubic feet a second. Continuous power development under this plan would, therefore, be based on 300 cubic feet a second at Mount Morris, something more than this at Geneseo and York, and on 600 cubic feet a second at Rochester. On this basis of computation it appears that the total permanent, continuous power to be realized from a reservoir storing 7,370,000,000 cubic feet and located in the Mount Morris Gorge would be 18,327 gross horsepower.

In regard to the increase in water power, the effective value of the storage will be the amount of permanent power above the low-water

power of the stream. As already stated, the total permanent power for the unregulated stream is 6,727 gross horsepower. The gain due to the storage is, therefore, 11,600 gross horsepower. Assuming a price of $10 per gross horsepower, we reach an annual return from the increased power of $116,000; but the Mount Morris reservoir is estimated to cost $2,785,000. If we assume the project carried out by a private company, with money at 5 per cent, the annual interest on the investment is $139,250—a sum $23,250 in excess of the probable annual income when all the power created shall have been brought into use; but there should be a sinking, maintenance, and repair fund of at least $25,000 a year, in order to repay the principal investment, which if taken into account increases the probable deficiency to $48,250 a year. It must be concluded, therefore, that with the present understanding as to the minimum run-off of Genesee River the project of a storage reservoir in Mount Morris Canyon, storing approximately 7,370,000,000 cubic feet of water, at a cost of $2,785,000, is commercially impracticable.

If we consider the Portage project in its financial aspects, where it is proposed to construct a reservoir storing 15,000,000,000 cubic feet of water, at a cost of $2,600,000, we reach the following results:

The total fall from just above the upper fall at Portage to the mean level of Lake Ontario is 785 feet, of which the greater portion is available for the development of water power. Without going into detail, we may place the permanent, continuous gross horsepower of the river, with a storage of 15,000,000,000 cubic feet at Portage, at the following figures:

Power of Genesee River after construction of reservoir at Portage.

	Gross horsepower.
Portage to Mount Morris	25,924
Mount Morris	835
Geneseo and York	624
Rochester	29,840
Total	57,223

Deducting from 57,223 gross horsepower the present permanent power of 6,727 gross horsepower, we have 50,496 gross horsepower as the net increase in the permanent water power of the stream due to the construction of the Portage reservoir. At $10 per gross horsepower, as before, the annual income when the power is utilized amounts to $504,960. The estimated cost of producing this vast increase in power is $2,600,000. Assuming an interest rate of 5 per cent, the annual interest is $130,000; adding to that amount $25,000 for sinking fund, maintenance, and repairs, the total annual expense becomes $155,000. The difference of $349,960 is the net annual income.

As already shown, when interest is taken into account, the Mount Morris project becomes commercially impracticable. The Portage project, on the other hand, shows an annual income, above interest account, sinking fund, maintenance, and repairs, of $349,950, which.

capitalized at 5 per cent, represents $6,999,000. If we assume 4 per cent, the capitalization of the annual income may be expected ultimately to represent $8,748,750.

SUMMARY.

Lack of space prevents discussion in greater detail of the Genesee River storage project, and the following summation is presented as embodying the main points of the discussion:

(1) Of the several available sites for reservoirs on Genesee River that at Portage is preferable to others, because it affords the largest storage at the smallest cost per unit volume.

(2) Serious floods have occurred a number of times in Genesee River at Rochester, the most serious being that of March, 1865. At least $1,000,000 loss resulted from that flood. The flood in April, 1896, was nearly as severe as the flood of March, 1865, although, as the river channel was clear, very little damage ensued.

(3) As the result of three years' measurements of Genesee River, it is determined that the minimum flow of the stream may for the entire year be as low as 6.67 inches on the watershed.

(4) A study of existing conditions shows that the Genesee River drainage area has been nearly denuded of forests, and hence that severe spring floods are likely to be frequent. For the same reason the summer flow of the stream is less than formerly.

(5) As a tentative conclusion, based on the data at hand, it may be said that deforestation of a drainage area may tend not only to increase floods somewhat, but to decrease materially the amount of the annual run-off.

(6) A comparison of the conditions existing on the drainage area of Genesee River with those of the upper Hudson, which is still largely in forest, shows less run-off under given conditions from the Genesee than from the Hudson, thus indicating the probable effect of the forest in increasing the run-off.

(7) As regards the upper Genesee drainage area, the forest has been removed by landowners who have commercially profited by such romoval; the effect, however, has been to injure permanently every riparian owner on the stream; hence it is proper that the State should spend money either in partially reforesting the area or in constructing river regulation works. The latter is preferable, because the benefits can be realized in a few years. If the State does not desire to construct such works, there should be no obstacles interposed to their construction by a private company.

(8) The proposed Portage reservoir will impound 15,000,000,000 cubic feet of water, at an estimated cost of $2,600,000, or at a cost of $173.33 per million cubic feet stored. It affords a permanent, continuous power above the present low-water flow of the stream of 50,496 gross horsepower, while the reservoir at Mount Morris affords *only* 11,600 horsepower above the present low-water power of the *stream.*

) Based on manufacturers' ratings, the present total developed ›r power of Genesee River from Portage to Rochester, inclusive, 9,178 net horsepower; or, basing the amount of water power on manufacturers' ratings of water required, and assuming 75 per ; efficiency on the wheels, the total power is 17,248 net horse- er, of which 16,683 net horsepower is within the limits of the city Rochester.

0) The enlarged Erie Canal will require 80 cubic feet of water second from Genesee River for every month of the navigation on except May, and in that month a mean of 177 cubic feet per nd.

1) The present extreme low-water power of Genesee River at hester is 5,046 net horsepower, of which one-half, or 2,523 net ıepower, will be rendered very much less valuable to its owners ıuse of the taking for the enlarged Erie Canal of 80 cubic feet per nd in every month of the navigation season and 177 cubic feet second in May.

WATER STORAGE ON HUDSON RIVER.

s noted in Part I of this paper, on page 33, Hudson River is divided ıe Troy dam into the upper or water-power section and the lower dal portion. The proposed reservoirs are, as a matter of course, ıe upper section, that above Troy.

EARLY SURVEYS.

he project for constructing storage reservoirs on the upper Hudson been agitated for many years, the first surveys for this purpose ing been made in 1874. In that year Prof. F. N. Benedict con- ted surveys, and in his report proposed an extensive system of res- ›irs. The chief interest attaching to this report is the proposition he part of Mr. Benedict to build storage reservoirs at Blue Moun- , Racket, Forked, Beach, and Long lakes, and divert the water ed on these several lakes from their natural drainage into Racket ›r, to the south, thus making them artificially tributary to Hud- River. In proposing this diversion, Mr. Benedict apparently med that the State, in its sovereign capacity, could divert waters ı one drainage basin to another without regard to the rights or .es of the riparian owners.

addition to the lakes already enumerated, which are naturally ıtary to Racket River, Mr. Benedict proposed to make reservoirs ıe following lakes and ponds in the upper Hudson drainage area: nd Pond, Catlin Lake, Rich Lake, Harris Lake, Lake Henderson, comb Lake, Lower Works reservoir, Chain Lakes, Goodenow l, Goodenow River reservoir, South Pond, Clear Pond, Slim l, Ackerman Pond, Perch Pond, Trout Pond, Lake Harkness, ld Lake, First Sergeant Pond, Third Sergeant Pond, Plumley

Pond, Moose Pond, and Cary Pond. The total storage to be furnished by the entire system of reservoirs is placed at 18,419,781,000 cubic feet. The total cost of the proposed reservoirs was placed by Mr. Benedict at about $265,000, or, including the diversion canal and improvements at Long Lake, at a total of about $460,000. The dams

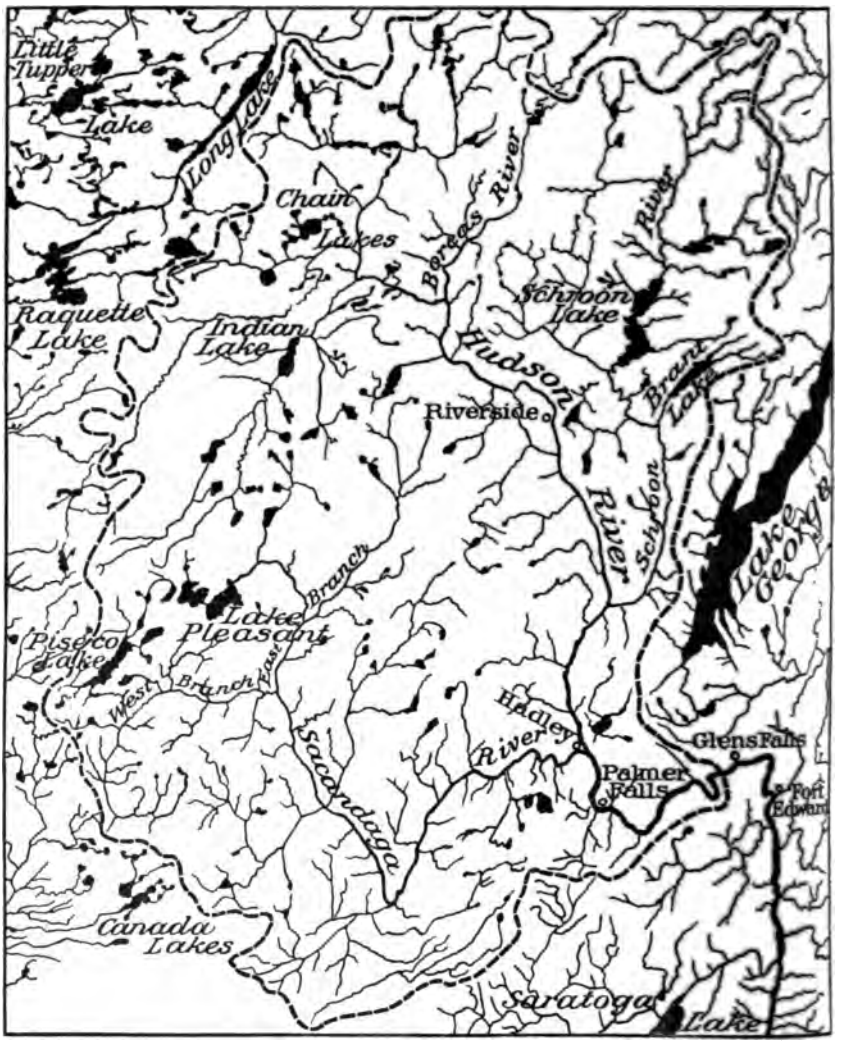


FIG. 1.—Map of drainage area of Hudson River above Glens Falls.

proposed were to be constructed of timber, very much after the plan of the timber dams still constructed by the lumbermen in this region, as shown on Pl. III.[1]

[1] *For further particulars of Mr. Benedict's reservoir system, see Report on a Survey of the Waters of the Upper Hudson and Racket Rivers in the Summer of 1874, with Recommendations Increasing the Supply of Water for the Champlain Canal and Improving the Navigation of the Hudson River*, by F. N. Benedict, Ass. Does. (1875), Vol. I, No. 6, p. 85.

A. LUMBERMAN'S DAM ON CEDAR RIVER

B. LUMBERMAN'S DAM ON INDIAN RIVER

In 1874, when Mr. Benedict prepared his report, the demands for water upon Hudson River were far less extensive than at present, and even in 1882 the total water power of the stream was, according to the statistics of the Report on the Water Power of the United States, Tenth Census, only 12,894 horsepower, while in 1895 the total horsepower was 43,481. Taking into account additional wheels set in the last two years, as well as the extensive development of the Hudson River Power Transmission Company now in progress 3 miles below Mechanicville, it is probable that early in 1898 there will be wheels set on Hudson River capable of furnishing, at full capacity, not far from 55,000 horsepower. This great development has led to a very strong demand in the last few years for increased flow during the low-water period.

RECENT INVESTIGATIONS.

In 1895 a survey of the upper Hudson Valley was authorized with the view of determining what lakes and streams may be improved, and the water stored and diverted, in order to provide for the enlargement of Champlain Canal; for restoring to the water of Hudson River at or below Glens Falls the water diverted therefrom for canal purposes, and for improving the navigation of the lower Hudson River. The Hudson River naturally divides into two sections—the upper and the lower—at the Troy dam, which is the head of the lower or tidal section. The proposed reservoirs are all in the upper section, above Troy.

When one considers the scope of the investigation it may be readily seen that the studies must necessarily be of rather wide range. Special consideration should be given the following topics:

(1) The area of the several subdivisions of the drainage area, together with the locations and extent of the reservoir sites, and the total area from which the run-off can be controlled.

(2) The rainfall and mean temperature of the tributary region, as well as its physical characteristics, the relative amounts of timber and cleared area, etc.

(3) The actual run-off of the stream from the known area for a series of years, and a deduction therefrom by comparison with the rainfall and temperature records of the amount which may be stored in the year of minimum rainfall; also the relation which the run-off in the year of minimum precipitation bears to what may be expected in the average year, and a deduction therefrom of the proper height of flow lines for full-capacity development.

(4) The areas of the reservoirs and the losses therefrom by evaporation which may be reasonably expected, with the amount of effective storage which may be gained by the reservoir system when developed to full capacity.

(5) The amount of water now diverted for the use of Champlain

Canal, and the amount to be diverted for such use when the enlargement is completed; also the proper method of managing the system of reservoirs in order to secure the best results to the canal, the navigable section, and the water power.

(6) The amount of water power now in use on the stream and the effect of the present and future diversion.

(7) The regimen of the tidal section, and the effect of the unregulated fresh-water flow and of the construction of the system of impounding reservoirs.

(8) The cost of the reservoirs and the relation which the actual cost bears to the amount of storage gained. This latter element determines the commercial feasibility of the project.

RESERVOIR SITES ON SACUNDAGA, MAIN HUDSON, AND SCHROON RIVERS.

The surveys, so far as carried, indicate that economical reservoirs controlling the entire drainage area to full capacity in the year of minimum rainfall may be constructed in the Sacundaga, main Hudson, and Schroon valleys, as shown in the following paragraphs.

The Sacundaga River has, as already stated, a total drainage area above its mouth at Hadley of 1,040 square miles. The catchment areas of reservoir sites on Sacundaga River, in square miles, are as follows:

Catchment areas of reservoir sites on Sacundaga River.

	Square miles.
Lakes Pleasant and Sacundaga	45
Piseco Lake	55
Arietta flow	40
Miscellaneous	50
Total	190

The main Hudson or North River has a total drainage area above Hadley, not including Schroon River, of 1,092 square miles. Of this area the portions shown below may be developed to full capacity in the year of minimum rainfall:

Catchment area of reservoir sites on main Hudson River.

	Square miles.
Thirteenth Pond	14
Chain Lakes	58
Catlin Lake	35
Lakes Rich, Harris, and Newcomb, and the Goodenow flow	83
Lake Henderson	18
Lake Sanford and the Tahawus flow	67
Boreas River and Boreas Pond	45
Cedar River	58
Indian Lake	146
Total	514

Schroon River has a total drainage area above its mouth of 550 quare miles. The topography of the Schroon area is such as to dmit of two distinct lines of treatment—either to construct one large lam at Tumblehead Falls, about a mile below South Horicon, or to construct a series of 16 to 18 small dams at various points in the area. In either case it is possible to control substantially the full flow of the 502 square miles above Tumblehead Falls, and the decision of which is better will turn chiefly on the question of relative cost, the estimate taking into account the fact that it will cost much more to operate a large number of reservoirs than to operate one.

Catchment area of small reservoirs on Schroon River.

	Square miles.
Minerva Brook at Olmsteadville	43.4
Hewett Pond	2.5
Loon Lake	11.6
Friend Lake	4.9
Elk Lake	15.9
Clear Pond	2.3
New Pond	1.7
Deadwater Pond	18.9
Hammond Pond	11.4
Dudley Pond	3.9
Overshot Pond	4.9
Paradox Lake	31.6
Paragon Lake	5.6
Crane Pond	7.2
Pharaoh Lake	8.3
Grant Lake	38.7
Valentine Pond	6.7
chroon Lake at Starbuckville	259.2
Total	478.7

The area between Starbuckville and Tumblehead Falls not available ith the system of small reservoirs is 23.3 square miles.

The total controllable area of the upper Hudson, with the system of nall reservoirs in the Schroon Valley, is as follows:

Total catchment area of upper Hudson River.

	Square miles.
acundaga Valley	190
udson above Hadley	514
chroon Valley	479
Total	1,183

With one large reservoir in Schroon Valley the catchment area is icreased to 1,206 square miles.

The system of small reservoirs outlined in the foregoing is estimated store 15,330,000,000 cubic feet, at a cost of $1,172,500; hence the

cost per million cubic feet stored becomes $76.48. These figures, however, do not take into account the annual cost of maintenance and operation, which may be placed at $30,000 per year, and capitalized at 5 per cent is equivalent to a permanent investment of $600,000. Adding $600,000 to $1,172,500 gives a total permanent investment of $1,772,500.

A general estimate of the cost of the single large reservoir in Schroon Valley shows that with a dam at Tumblehead Falls 59 feet in height there would be impounded 15,925,000,000 cubic feet. The preliminary estimate indicates a total cost of $840,000, and a later survey indicates about $1,000,000. The final revision of the estimate on completion of the investigation may show a somewhat larger figure than this. Even if the cost were to be $1,100,000, it would still be exceedingly cheap storage, the cost for 15,925,000,000 cubic feet being on this basis only $69.14 per million cubic feet stored.

The dam at Tumblehead Falls would be located just below the outlet of Brant Lake, the elevation of the water surface of which is 801 feet. The flow line of the proposed reservoir has been placed at an elevation of 840 feet, thus giving a depth of 39 feet over the surface of Brant Lake, a depth of 33 feet over the surface of Schroon Lake, and a depth of 20 feet over Paradox Lake. With the reservoir full or nearly full, there would be continuous navigation from the head of Brant Lake to the head of Paradox Lake of about 35 miles. The villages of South Horicon, Bartonville, Starbuckville, and parts of Pottersville and Chester are within the flow line of this reservoir.

As another large reservoir to be built on the head waters of the Hudson, Indian Lake reservoir may be mentioned. As shown on page 128, the total controllable area at this lake is 146 square miles, which is capable of furnishing, in the year of minimum run-off, 4,468,000,000 cubic feet. To provide this storage, a dam raising the water surface 23 feet above the present dam is required. The storage of the present dam 10 feet in height is estimated at 800,000,000 cubic feet. In the spring of 1897 a private company, known as the Indian River Company, was organized to construct the Indian River reservoir. This company proposes to construct a masonry and concrete dam to the height of 23 feet above the old timber dam. The work is now in process of construction. The clearing of the margins of the lake includes the cutting of about 1,160 acres of timber.

The Indian River Company was the owner of considerable land bordering on and in the vicinity of Indian Lake. In consideration of the transfer of townships 15 and 32 of the Totten and Crossfield purchase to the State forest preserve board, that board has, in effect, purchased the dam in process of construction, which, with the clearing of the margins, is estimated to cost about $100,000. The forest preserve board stipulated that the superintendent of public works

re the right to draw water from the reservoir when necessary for supply of Champlain Canal, the balance of the storage to be used the Indian River Company for increasing the low-water flow of dson River for the benefit of the water power. At a total cost $100,000 the cost per million cubic feet stored becomes $22.38. king into account the quantity of water stored at Indian Lake, this ist be considered a very cheap reservoir.[1]

Piseco Lake may also be referred to as another large reservoir which y be constructed on the upper Hudson at low cost. It is estimated t a storage of 1,725,000,000 cubic feet may be made, at an expend- re of $70,000, or at an average cost per million cubic feet stored $40.

Without going further into detail, the following may be given as the proximate storage of the entire upper Hudson system, worked out date.[2]

Approximate storage of upper Hudson River.

	Cubic feet.
rage of Sacundaga and main Hudson River drainage areas, not ncluding Boreas River reservoir, Boreas Pond, Indian Lake, and iseco Lake	14,364,000,000
reas River reservoir and Boreas Pond	1,111,000,000
lian Lake	4,468,000,000
eco Lake	1,725,000,000
iroon Valley	15,925,000,000
dley	4,000,000,000
nklinville	4,000,000,000
Total	45,593,000,000

This storage is considered sufficient, in conjunction with the natural w of the unregulated portion of the river, to maintain at Mechan- ville a flow of at least 4,500 cubic feet per second during the entire ar.

In addition to the reservoirs named above, the general investiga- ns indicate that there is, possibly, an opportunity to make a large ervoir on Sacundaga River by the erection of a dam about 20 feet height at Conklinville, where there is a reach of river extending 24 les to Northville. The available storage of such a reservoir is taken about 4,000,000,000 cubic feet.

There is also an opportunity to construct on the main Hudson at dley, just above the mouth of the Sacundaga, another reservoir of out 4,000,000,000 cubic feet capacity, at a point where the natural ditions for constructing such a reservoir are considered very good. the site of the proposed dam the river shows a granitic rock bottom

---

[1] This dam was completed in 1898.

[2] For full details the reader is referred to the original reports on the upper Hudson storage veys, in the Annual Reports of the State Engineer and Surveyor for the fiscal years ending ember 30, 1895 and 1896.

with precipitous banks nearly 40 feet in height and about 100 feet apart. The material for a permanent stone dam exists in the vicinity, with an opportunity to construct a wasteway over natural rock at one side.

Inasmuch as all the storage except that of the Sacundaga area would pass through the Hadley reservoir, its construction would simplify the management of the system very greatly. In the summer season, as long as there is any storage above to be drawn upon, this reservoir could be kept nearly full and just the right quantity drawn out from day to day to keep the river at the assumed flow of 4,500 cubic feet a second at Mechanicville.

EFFECT OF PROPOSED STORAGE ON RIVER FLOW.

The foregoing quantities of storage have been fixed upon on the basis that the water yield of the year of minimum stream flow will furnish a storage of at least 12 inches, the flow line of the reservoirs themselves being located with reference to holding back 13.5 inches. If, however, one examines the tables of run-off of the Hudson at Mechanicville, given in Part I of this paper, on page 82, and of precipitation in the watershed, given on the next page, it is seen that much greater yields can be expected in an average year. From this point of view, it may be asked, Why not make the reservoirs somewhat larger than merely sufficient for the wants of the year of minimum flow and carry some water over from one year to another, thus more nearly attaining an absolute regulation of the river—not for a single year, but for a series of years? The chief objection to this method of procedure is that experience with other large reservoir systems is against other than a moderate development on this line, it having been repeatedly found that however high the flow line, reservoirs are likely to be nearly empty at the beginning of the storage period of the minimum year. Experience indicates that the rainfall and stream flow move in cycles, there being in each cycle several successive years of flow above the average. The demands for water tend to increase during the years of plenty, until those in charge apparently forget there will ever be a deficiency. The best practice, therefore, is to locate the flow line with reference to about the minimum yield, thus forcing an economy in the use of water from the beginning. By proceeding in this way provision may be made for carrying over moderate quantities of water from the latter end of the year more effectually than in any other way.

A. THE GEORGE WEST PAPER MILL, ON HUDSON RIVER AT HADLEY, NEW YORK.

B. THE HUDSON RIVER PULP AND PAPER COMPANY'S MILLS AT PALMER FALLS, ON HUDSON RIVER, NEW YORK.

Mean precipitation on the upper Hudson watershed.

[In inches.]

th.	Albany.	Glens Falls.	Keene Valley.	Mean of Albany, Glens Falls, and Keene Valley.	Western Massachusetts.	Northern Plateau	Lowville Academy.	Johnstown Academy.	Cambridge Academy.	Fairfield Academy.	Granville Academy.	Mean of all.
er	2.71	3.19	2.75	3.01	4.31	3.57	2.14	2.91	2.36	2.78	2.48	2 92
r.....	2.75	3.16	3.00	3.03	3.36	3.88	2.35	3.30	3.36	2.69	2.08	2.99
ry	2.49	3.03	2.22	2.64	2.83	3.60	2.43	2.86	2.61	2.06	1.42	2.56
.....	2.72	2.72	2.24	2.50	2.94	2 52	1.78	3.63	2.12	2.36	1.74	2.48
.....	2.80	2.15	2.09	2.14	2.02	2.39	1.91	2.93	3.36	2.53	2.13	2.43
.....	3.62	3.10	3.05	3.14	3 17	4 46	2.79	3.45	3.65	3.04	3.47	3.38
	17.09	*17.35*	*15.35*	*16.47*	*18.63*	*20.42*	*13.40*	*19.08*	*17.46*	*15.46*	*13.32*	*16.76*
.....	4.07	2.83	2.89	2.99	3.24	3.88	3.42	4.20	4.66	4.29	3.21	3.67
.....	4.29	3.25	3.61	3.53	3.44	3.83	3.67	4.01	3.91	4.21	3.63	3.78
.....	3.96	4.17	4.21	4.20	3.91	5.07	2.85	3.14	3.98	3.66	2.97	3.79
	12.32	*10.25*	*10.71*	*10.75*	*10.59*	*12.78*	*9.94*	*11.35*	*12.55*	*12.16*	*9.81*	*11.25*
ber ...	3.43	2.96	3.00	3.13	3.51	3.57	2.84	2.87	3.27	3.08	2.67	3.12
.....	3.58	2.36	2.49	2.60	3.41	3.06	3.29	3.20	3.60	3.56	2.90	3.15
ber ...	3.08	3.24	3.56	3.26	2.84	3.46	2.94	3.33	3.29	2.46	2.88	3 11
	10.09	*8.56*	*9.05*	*8.99*	*9.76*	*10.09*	*9.07*	*9.49*	*10.16*	*9.11*	*8.45*	*9.38*
tal ...	39.50	36.16	35.11	36.22	38.98	43.29	32.41	39.92	40.17	36.73	31.58	37 39

gures in the above table are obtained by averaging the results obtained at Albany from 895; at Glens Falls, from 1879 to 1895; at Keene Valley, from 1879 to 1895; in western Massetts, from 1887 to 1895; in Northern Plateau, from 1889 to 1895; at Lowville Academy, from 1848; at Johnstown Academy, from 1828 to 1845; at Cambridge Academy, from 1827 to Fairfield Academy, from 1828 to 1849; at Granville Academy, from 1835 to 1849; the mean ny, Glens Falls, and Keene Valley, from 1879 to 1895. Although the foregoing figures are 'en in detail, later studies indicate that the mean rainfall of the Northern Plateau as by the State meteorological bureau is the best rainfall record to apply to the upper Hud-

proposed regulation of Hudson River, as thus far planned, is arranged on the basis of maintaining a flow of at least 4,500 feet per second at Mechanicville, where, as has been seen, the ıge area is 4,500 square miles, such a regulation being equivalent ɔducing at Mechanicville a constant flow of 1 second-foot per ɜ mile. The relation of such regulated flow to the natural ulated flow may be seen by studying the diagrams of flow of ɔn River at Mechanicville given in Part I, on page 81.

'egards the change in the regimen of the stream due to storage, y be remarked that the reservoirs have been designed on the ɔf giving to the stream at least 0.5 inch on the drainage area per . This amounts to 0.45 cubic foot per second per square mile ıonth of thirty days; it is not, however, intended to state that the river will ever be as low as 0.45 cubic foot per second per square ɔr, what is the same thing, as low as 2,025 cubic feet per second ɜhanicville, but only that those tributary streams on which the e reservoirs are located may be down to this figure. With 0.45

cubic foot per second per square mile always flowing away from the controlled drainage area, the natural flow of the unregulated portion will usually furnish an additional amount sufficient to keep the river, during the storge period, up to nearly the assumed 4,500 cubic feet at Mechanicville; or in case of extreme low water in winter other reservoirs may be relied upon to assist in the manner already pointed out.

On the basis of 12 to 14 inches available storage, there may be, with 0.5 inch per month always going to the stream, a possible total requirement for the year of from 15 to 18 inches.

The table in Part I of this paper, on page 82, shows that the total flow for the water year 1895 was only 17.46 inches, or in that year there might have been a shortage if the reservoir system had been in operation of perhaps 0.5 inch. Any such shortage would necessarily have been carried over from the year 1894, when, in November, there was a run-off of 1.58 inches. Allowing 0.5 inch to the stream, from the November rainfall alone there would have been 1.08 inches remaining in the reservoirs to be carried over to 1895.

SUMMARY.

In conclusion, it may be said that it is entirely feasible to construct a system of reservoirs in the upper Hudson Valley, and such system may be designed with reference to the full capacity storage of at least 1,300 square miles of area, or 47 per cent of the total area above Glens Falls. Such control would result in the material reduction of floods at Glens Falls and other points.

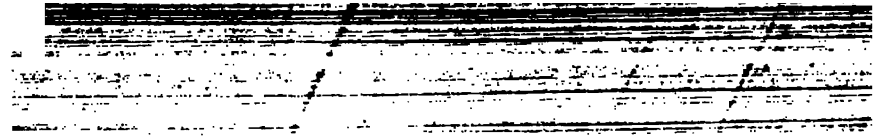
The proposed total storage of 45,593,000,000 cubic feet would maintain 4,500 cubic feet per second flow, as well as supply the other necessary demands, in the driest season of the gaging period. The discharge measurements show that whereas the minimum unregulated flow at Glens Falls is as low as 900 cubic feet per second for a monthly mean, with the storage carried out, the probable monthly mean flow at Glens Falls will be at least 3,000 to 3,600 cubic feet per second. The minimum regulated flow of 4,500 cubic feet per second at Mechanicville will increase the low-water depth in Hudson River at Albany about 1.5 feet.

The diversion of water for the use of Champlain Canal is an injury to the water power at Glens Falls and lower points on the river. Since water power is much cheaper than steam power, the taking of the water of the river away from the manufacturers is a serious matter. In the fourteen years from 1882 to 1895 the use of water power on Hudson River has increased from a total of 12,894 net horsepower in 1882 to 43,481 net horsepower in 1895, an increase of 237 per cent.

The upper Hudson storage system is estimated to cost from $60 to $70 per million cubic feet stored, a sum considerably less than the cost of many other systems.

A. GLENS FALLS PAPER MILLS AT FORT EDWARD, NEW YORK.

B. DAM OF THE HUDSON RIVER POWER TRANSMISSION COMPANY DURING CONSTRUCTION, OCTOBER, 1897



DEVELOPMENT OF WATER POWERS.

POWER DEVELOPMENT AT NIAGARA FALLS.

The possibility of water-power development at Niagara Falls has ttracted attention for many years, the first utilization there having een made in 1725, when the French erected a sawmill near the point here the Pittsburg Reduction Company's upper works now stand for ıe purpose of supplying lumber for Fort Niagara. Between 1725 and ıe early years of the present century little is known of the use made f Niagara Falls power further than that sawmills were in operation ıere during the whole period. In 1805, however, Augustus Porter uilt a sawmill on the rapids, and in 1807 Porter & Barton erected a ristmill. In 1817 John Witmer built another sawmill at Gill Creek, ıd in 1822 Augustus Porter built a gristmill along the rapids above e falls. From that year to 1885, when the lands along the river ere taken for a State park, a considerable amount of power was veloped by a canal which took water out of the river near the head the rapids and followed along the shore nearly parallel with the nk of the river. Mills were built between this canal and the river, d a part of the 50-foot fall between the head of the rapids and the ink of the American Falls was thus utilized. A paper mill was ilt on Bath Island at an early date.

In 1842 Augustus Porter, one of the principal mill owners at agara Falls, proposed a considerable extension of the then exist- g system of canals and races, and in January, 1847, in connec- on with Peter Emslie, he published a formal plan which became e subject of negotiations with Walter Bryant and Caleb S. Wood- ıll. An agreement was finally reached by which they were to nstruct a canal and receive a plot of land at the head of the nal, having a frontage of 425 feet on Niagara River, together with right of way 100 feet wide for the canal along its entire length of 400 feet, and about 75 acres of land near the terminus, having a ontage on the river below the falls of nearly a mile. The canal nstructed under this agreement passes through what is now the ost thickly settled part of the city of Niagara Falls.

Ground was broken by Messrs. Bryant & Woodhull in 1853 and e work carried on for about sixteen months, when it was suspended r lack of funds. Nothing further was done until 1858, when Stephen len carried the work forward for a time; later, in 1861, Horace H. ıy took up the matter and completed a canal 36 feet wide, 8 feet ep, and 4,400 feet long, by which the water of the upper river was ought to a basin near the brink of the high bluff of the lower river d at an elevation of 214 feet above the lower river. Upon the margin this basin various mills have been constructed, to the wheels of which ter is conducted from the canal and discharged through the bluff o the river below. The first mill built on this hydraulic canal was a all gristmill, erected by Charles B. Gaskill in 1870 on the site of present large flouring mill of the Cataract Milling Company.

NIAGARA FALLS HYDRAULIC POWER AND MANUFACTURING COMPAN

In 1877 the hydraulic canal and all its appurtenances were pu chased by Jacob F. Schoellkopf and A. Chesbrough, of Buffalo, w organized the Niagara Falls Hydraulic Power and Manufacturi Company, of which Mr. Schoellkopf is still president. The followi is a list of companies either now supplied or to be supplied with pow by the Niagara Falls Hydraulic Power and Manufacturing Compan

Power furnished by Niagara Falls Hydraulic Power and Manufacturing Compa

WATER POWER.

Company.	Business.	Horse-power.
Central Milling Co	Flouring mill	1,000
N. Wood Paper Co	Paper and pulp	500
Schoellkopf & Mathews	Flouring mill	900
Pettebone Cataract Manufacturing Co.	Paper and pulp	2,000
Cataract Milling Co	Flour	400
Niagara Falls Waterworks		200
Thomas E. McGarigle	Machine shop	25
Cliff Paper Co	Paper and pulp	2,500
Total		7,525

ELECTRIC POWER.

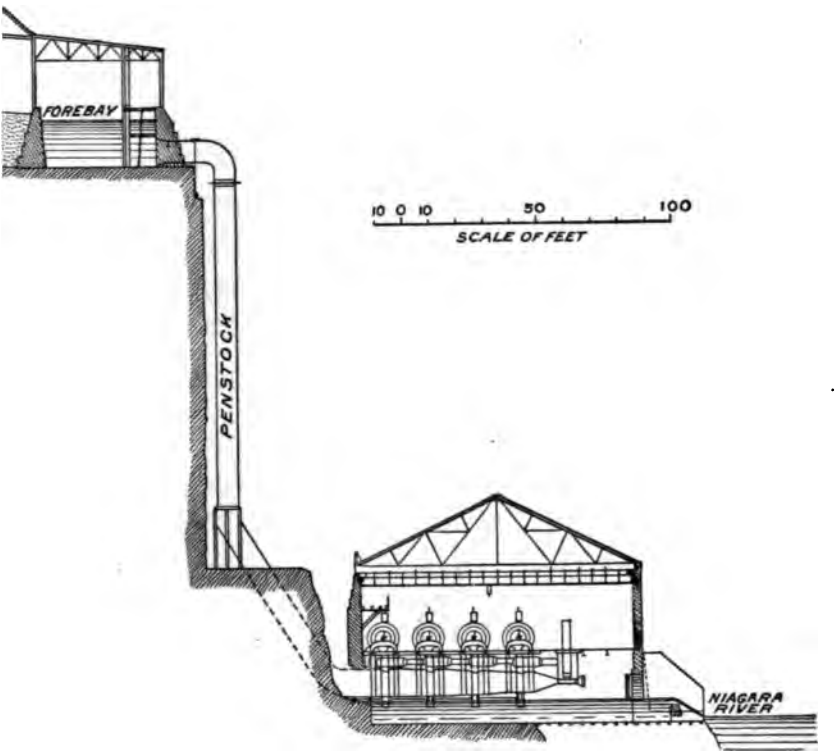
Pittsburg Reduction Co	Aluminum	3,500
Niagara Falls and Lewiston R. R. Co		400
Cliff Paper Co	Paper and pulp	300
Lewiston and Youngstown R. R. Co.		200
Buffalo and Niagara Falls Electric Light and Power Co.	Light and power	600
Niagara Falls Brewing Co		150
Rodwell Manufacturing Co	Silver plating, etc.	75
Sundry small customers in Niagara Falls.		100
Francis Hook and Eye Co	Hooks and eyes	15
Kelly and McBean Aluminum Co	Aluminum	15
The National Electrolytic Co		1,000
Total		6,355

MECHANICAL POWER FURNISHED ON SHAFT.

Oneida Co., Limited	Silver-plated ware and chains.	300
Carter-Crum Co	Check books	60
Total		360
Grand total		[illegible]

POWER HOUSE OF THE NIAGARA FALLS HYDRAULIC POWER AND MANUFACTURING COMPANY.

The contract made in 1852 between Augustus Porter, Walter yant, and Caleb J. Woodhull only conveyed lands to the edge of e high bank of Niagara River, but did not include the talus or slope tween the edge of the high bank and the river, and only granted e right to excavate 100 feet down the face of the bank. In 1852, hen this contract was made, the use of water power under higher ads than 100 feet was, so far as the United States was concerned, tirely unknown. Until recently the mills at Niagara Falls have ot attempted to use more than 50 or 60 feet head; hence it resulted at although the capacity of the Niagara Falls Hydraulic Power and



G. 2.—General plan of development of the Niagara Falls Hydraulic Power and Manufacturing Company.

anufacturing Company's canal, as at first constructed, was sufficient, y development of the whole head, to produce about 15,000 horse-ower, under the original agreements its capacity was exhausted when bout 7,000 horsepower was produced.

In 1892 the Niagara Falls Hydraulic Power and Manufacturing ompany began an enlargement and improvement of its canal. The an adopted was to widen the original channel at one side to 70 feet, d make the new part 14 feet deep. This work is cut entirely rough rock, below the water line. The enlargement of one side as completed in 1896. The canal as enlarged to date has a capacity

of about 3,000 cubic feet per second, giving, under present conditions, a total of from 40,000 to 50,000 horsepower, the cross section being about 400 square feet.

This company has a grant from the State of the right to draw from Niagara River as much water as can be taken through a canal 100 feet wide and 14 feet deep. Work on further enlargement is now in progress, and it is expected, within a year or two, to have a total of about 675 feet area of water section.

To July 1, 1897, about 100,000 cubic yards of material had been taken out at a cost of $250,000, the average cuttings in the original canal from the surface of the ground to the surface of the water being about 8 feet.

The development now in process by this company is very interesting. A bulkhead is located at the top of the high bank with a fore bay back of it connected with the main hydraulic canal by a short-branch canal. From the fore bay a large penstock leads vertically down the cliff to a power house located directly on the shore of the lower river. In this power house horizontal turbine water wheels are placed, with dynamos directly connected, the power therefrom being transmitted either to the mills on the bluff above or to establishments at a distance. (See Pl. VII and fig. 2.) This company expects soon to transmit several thousand horsepower to Buffalo.[1] Without taking into account the cost of water in the canal, the cost of the development of power in the way in which it is now being developed by this company may be placed at $35 per horsepower.

NIAGARA FALLS POWER COMPANY.

The Niagara Falls Power Company has developed an extensive plant on quite different lines from that of the Niagara Falls Hydraulic Power and Manufacturing Company. In 1883 to 1885 Thomas Evershed, who was at that time division engineer of the western division of the New York State canals, was called on to survey Niagara Falls Park Reservation, as provided for by act of the legislature. This led Mr. Evershed to spend considerable time at Niagara Falls, during which he conceived the project of constructing a tunnel to begin at the level of the lower river and extend under the city of Niagara Falls for a distance of about 2½ miles. (See fig. 3.) This tunnel, as proposed, was to be generally parallel to the Niagara River, but at some little distance from it. At its head and at various points along the river from above Port Day it was proposed to construct branch canals

[1] For further details of the Niagara Falls Hydraulic Power and Manufacturing Company, see (1) Power development of Niagara Falls, other than that of the Niagara Power Company, by W. C. Johnson: Trans. Engineers' Society of Western New York, Vol. I, No. 6 (Feb. 3, 1896). (2) Niagara Falls Hydraulic Power and Manufacturing Company's new work, by Orrin E. Dunlap: Electrical Engineer, Vol. XX (Dec. 4, 1895); (3) Old hydraulic power plant at Niagara Falls transformed for electrical transmission, by Orrin E. Dunlap: Western Electrician, Vol. XIX (Dec. 5, 1896); (4) Pulp mill of the Cliff Paper Company of Niagara Falls, New York, by W. C. Johnson: Trans. Am. Soc. Civil Eng., Vol. XXXII (Aug., 1894).

PENSTOCK OF THE NIAGARA FALLS HYDRAULIC POWER AND MANUFACTURING COMPANY.

ecting with the river and through which water could be taken, to scharged upon turbine wheels placed in vertical wheel pits and ected with the tunnel at various points.

e Niagara River Hydraulic Tunnel, Power and Sewer Company iagara Falls was incorporated in 1886 for the purpose of con- ting and operating, in connection with Niagara River, a hydraulic el or subterranean sewer for public use in the disposal of sewage drainage and for furnishing hydraulic power for manufacturing oses in the town of Niagara Falls. In consideration of the public ce of sewerage and drainage, this company was authorized to ire land by condemnation.

e general plan of development is described by Mr. Evershed in

FIG. 3.—Map of Niagara Falls and vicinity, showing location of the great tunnel. (From Cassier's Magazine, Vol. VIII, p. 183.)

port made July 1, 1896, in which he states that the main tunnel ld begin at a point on the lower river immediately north of the e reservation, with its mouth as low as high water below the falls ld permit. From this point to half a mile above Port Day it should e a rise of 1 foot in 100, or 52.8 feet per mile, and a section above t Day equivalent to a circle 24 feet in diameter, the tunnel gradu- diminishing in size in accordance with the number of mills empty- tail-water into it, until at the upper end it would have the same of cross section as the connecting cross tunnels.[1]

he matter remained in abeyance until 1889, when the Niagara Falls

e pamphlet, Water Power at Niagara Falls, prospectus of the Niagara River Hydraulic al, Power, and Sewer Company (1886).

Power Company was organized to carry out, in effect, Mr. Evershed's plan. The actual work of construction was undertaken by the Cataract Construction Company, composed of William B. Rankine, Francis Lynde Stetson, Pierpont Morgan, Hamilton McK. Twombly, Edward A. Wickes, Morris K. Jesup, Darius Ogden Mills, Charles F. Clarke, Edward D. Adams, Charles Lanier, A. J. Forbes-Leith, Walter Howe, John Crosby Brown, Frederick W. Whirtridge, William K. Vanderbilt, George S. Bowdoin, Joseph Larocque, John Jacob Astor, and Charles A. Sweet. This company has modified the original plans in some particulars, although the general scheme has been carried out.

The plan finally determined on comprised a surface canal 250 feet in width at its mouth on the river, 1¼ miles above the falls, extending inwardly 1,700 feet, with an average depth of 12 feet, and computed to furnish water sufficient for the development of about 120,000 horsepower. The masonry walls of this canal are pierced at intervals with inlets, guarded by gates. Some of these are used to deliver water to tenants who construct their own wheel pits and set their own wheels, while 10 of them are arranged on one side of the canal for the purpose of delivering water to the wheel pit of the Niagara Falls Power Company's power station, where dynamos, placed at the top of turbine vertical shafts, generate electricity for transmission. The wheel pit at the power station is 178 feet in depth and connected with the main tunnel by a short cross tunnel. The main tunnel as carried out has a maximum height of 21 feet and a width of 18.82 feet, making a net section of 386 square feet. The slope of this tunnel is 6 feet to the thousand.

The most careful consideration was given to the subject of the turbines to be used, as well as to the question of power transmission. In 1890 Edward D. Adams, who was then president of the company, established an International Niagara Commission, with power to offer $20,000 in prizes. This commission consisted of Sir William Thomson (now Lord Kelvin), Dr. Coleman Sellers, Lieut. Col. Theodore Turrettini, Prof. E. Mascart, and Prof. W. C. Unwin. Inquiries concerning the best-known methods of development and transmission of power in England, France, Switzerland, and Italy were made, and competitive plans were received from twenty carefully selected engineers and manufacturers of power plants in England, Europe, and America. These plans were submitted to the commission, which awarded prizes to those considered worthy. The most important result was the selection of the designs of Faesch and Piccard, of Geneva, for turbines computed to yield 5,000 horsepower each. Three wheels have been built from these designs and are now in place and regularly operated.

Without going into details of the electrical work, it may be stated that the Niagara Falls Power Company adopted the two-phase alternating current system as best adapted to its work. In the dynamos employed the field magnet revolves instead of the armature. As

A. POWER HOUSE OF THE NIAGARA FALLS POWER COMPANY

B OUTLET OF TUNNEL OF THE NIAGARA FALLS POWER COMPANY

advised by the company's electrical engineer, Prof. George Forbes, of London, three such dynamos, of 5,000 horsepower each, constructed by the Westinghouse Company, of Pittsburg, have been installed. During the summer of 1896 a transmission line was constructed from Niagara Falls to Buffalo, and since November of that year some of the street railways in Buffalo have been operated electrically by power from Niagara Falls.

According to a statement of William B. Rankine, secretary of the Cataract Construction Company, the power furnished or contracted for by the Niagara Falls Power Company July 1, 1897, was as follows:

Power furnished by the Niagara Falls Power Company.

HYDRAULIC POWER.

Company.	Business.	Horse-power.
Niagara Falls Paper Company	Paper	7,200
ELECTRIC POWER.		
Pittsburg Reduction Company	Aluminum	3,050
The Carborundum Company	Abrasives	1,000
Acetylene Light, Heat, and Power Company	Calcium carbide	1,075
Buffalo and Niagara Falls Electric Light and Power Company.	Local lighting	500
Walton Ferguson	Chlorate of potash	500
Niagara Electro-Chemical Company	Peroxide of sodium	400
Buffalo and Niagara Falls Electric Railway	Local railway	300
Niagara Falls and Suspension Bridge Railway Company.*a*	do	250
Buffalo Street Railway Company	22-mile transmission	1,000
Acetylene Light, Heat, and Power Company *b*	Calcium carbide	4,000
Mathieson Alkali Works *c*	Soda ash	4,000
Buffalo Street Railway Company		1,000
Buffalo General Electric Company *d*	Lighting	3,000
The Carborundum Company *e*	Abrasives	1,000
Niagara Falls Water Works Company		45
Power City Foundry and Machine Company		25
Albright and Wilson	Electro-chemicals	400
Total hydraulic power sold at Niagara Falls		7,200
Total electric power sold at Niagara Falls		14,545
Total electric power sold at Buffalo		5,000
Total		26,745

a All from October 1, 1896.
b From delivery, say, November 1, 1897.
c From June 1, 1897.
d From November 15, 1897.
e From June 1, 1897.

Recapitulation of the total power in use or furnished from Niagara Falls January 1, 1898, shows the following amounts:

	Horsepower.
Hydraulic power:	
Niagara Falls Power Company	7,200
Niagara Falls Hydraulic Power and Manufacturing Company	7,535
Electric power:	
Niagara Falls Power Company	19,545
Niagara Falls Hydraulic Power and Manufacturing Company	6,355
Mechanical power:	
Niagara Falls Hydraulic Power and Manufacturing Company	380
Total	40,965

The large demand for power from the Niagara Falls Power Company has necessitated the enlargement of the wheel pit and power house to more than three times their present capacity. The work upon this enlargement, sufficient to provide 35,000 additional horsepower, has been in progress since June, 1896, and is now approaching completion. With the completion of this extension the Niagara Falls Power Company will have available 50,000 horsepower; a portion of the increased power was ready for delivery January 1, 1898.

The 50,000 horsepower developed when the present extension is completed represents but one-half of the capacity of the present tunnel. This company has further secured the right of way for a second discharge tunnel, so that when the demand for power shall render it necessary the present plant may be duplicated, thus furnishing 200,000 horsepower in all. In addition to this large development on the American side, the Canadian Niagara Power Company, an allied corporation, now holds from the Canadian government an exclusive franchise granting to it the right to develop on the Canadian side at least 250,000 horsepower. The total possible power proposed to be developed in the future at Niagara Falls is about as follows:[1]

Power proposed to be developed at Niagara Falls.

	Horsepower.
Niagara Falls Power Company's present tunnel	100,000
Niagara Falls Power Company's second tunnel	100,000
Niagara Falls Hydraulic Power and Manufacturing Company's canal	150,000
Canadian Niagara Power Company's tunnels	250,000
Total	600,000

The developments now in progress at Niagara Falls are being carried out on very broad lines and probably furnish the best examples of modern hydraulic work. They certainly lead so far as the

[1] For an interesting discussion as to the effect of diverting large quantities of water from Niagara River for power purposes, see report of Clemens Herschel, made December 12, 1895, on the Diversion of Water from the Niagara River for Power Purposes by the Niagara Falls Hydraulic Power and Manufacturing Company and by the Niagara Falls Power Company, and the Unimportant Effect of such Diversion upon the River. Mr. Herschel bases his discussion on the data of the Lake Survey of an ordinary and usual flow of 265,000 cubic feet per second. Reasoning from this premise he concludes that even when 300,000 or 400,000 horsepower are in use the effect upon the depth of the river will be insignificant.

United States is concerned. A complete account of both works, giving details of all the engineering features, would make a large-sized monograph. From this point of view it is only possible to cite some of the sources of information.[1]

POWER PLANT AT MASSENA, ON ST. LAWRENCE RIVER.

Among the large power developments now under construction in the State of New York is that at Massena, on St. Lawrence and Grass rivers. According to statements made by John Bogart, consulting engineer, the power plant now under construction at Massena includes the excavation of a canal leading from St. Lawrence River to Grass River, a distance of 3 miles, the building of a power house, the installation of fifteen 5,000-horsepower electric generators, and the necessary equipment of turbine water wheels. The furnishing of the electric apparatus has been awarded to the Westinghouse Electrical and Manufacturing Company, and is said to be the largest contract for electric apparatus ever placed.

The plan of development adopted at Massena is to divert a portion of the water of St. Lawrence River from its natural channel by means of a canal, carrying it 3 miles across to Grass River, where, after operating turbines, it will pass by way of Grass River to the St. Lawrence at a point lower downstream. Just below where the canal takes water from St. Lawrence River, Long Sault Rapids are located, which have a fall of about 50 feet. Grass River runs nearly parallel to the St. Lawrence for several miles, flowing into the St. Lawrence a short distance below the foot of Long Sault Rapids. To the south of St. Lawrence River, and between it and the valley of Grass River, there is a comparatively level plateau.

The average width of Grass River from its mouth to above where the power canal will intersect it is from 250 to 300 feet, and its water surface, for this portion, is substantially on a level with the St. Lawrence below the rapids; hence the surface of Grass River at the point where the power canal strikes the stream is from 45 to 50 feet below the surface of the St. Lawrence at the head of the canal. The power station will be located on the north bank of Grass River, the tail-water dropping into that stream, which thus becomes, in effect, a tailrace for this power development. Making some allowance for increased depth of water in Grass River between the power station and its mouth, when receiving the tail water, and also some allowance

[1] The main facts in regard to the plant of Niagara Falls Power Company as herein embodied have been furnished by L. H. Groat, secretary of the company. For more extended information the reader is referred to (1) Cassier's Magazine, Vol. VIII (July, 1895), where may be found an account of nearly every phase of the Niagara Falls Power Company's development; (2) The Electrical World, Vol. XXX (Oct. 23, 1897), which may be consulted for a description of the extension of the wheel pit now in process; (3) Niagara Falls publication of the Niagara Falls Chamber of Commerce, issued in 1897; (4) the various numbers for 1897 of Greater Buffalo, a monthly publication devoted to promoting the prosperity of Buffalo and Niagara Falls. Engineering News and other technical journals may also be consulted.

for inclination of the water surface of the head canal, it is thought that an absolutely permanent power of about 40 feet will be obtained.

The work of constructing the head canal and preliminary work on the foundations of the power house are now under way. It is expected that the work will be completed in 1899. The canal will be 250 feet wide and 25 feet deep, this capacity being assumed as sufficient to furnish 150,000 horsepower. The work is being carried out by the St. Lawrence Power Company, which was organized under the laws of the State of New York, with a capital stock of $6,000,000. According to statements made, a large amount of power has been taken by an English syndicate, which intends to use it chiefly for electro-chemical manufacturing.

INLAND WATERWAYS.

TRADE AND COMMERCE OF HUDSON RIVER.

The importance of Hudson River as a great waterway of commerce is shown by Mr. Charles G. Weir in a report made in 1890. Aside from its own local trade the river absorbs all the traffic of the Erie, Champlain, Delaware, and Hudson canals, besides the great coal trade of the Pennsylvania Coal Company at Newburg and the Erie coal trade at Piedmont. The average season of navigation of the river is two hundred and forty days. The two principal industries on Hudson River, which add materially to the total tonnage, are ice and brick. The capacity of the ice houses on and near the river exceeds 4,000,000 tons, and the amount annually harvested is about 3,500,000 tons. The bricks manufactured on the river exceed 850,000,000.

The following statistics include the tonnage received at all points above Spuyten Duyvil Creek, and of the local shipment between points on the river. That shipped is credited only to the points from which it was shipped, no entry being made to the total tonnage of the amount received at local points from other local points. The total tonnage also includes all through freights shipped from points up the river that passed the mouth of Spuyten Duyvil Creek going south.

Tonnage and value of commerce on Hudson River above Spuyten Duyvil Creek.

Total tonnage of all shipping points on Hudson River during 1889, not including the tonnage coming through State canals (tons)	15,033,309
Value of same	$378,196,094
Total tonnage coming to and leaving tide water through State canals, 1889 (tons)	3,592,437
Value of same	$108,000,000
Increase of same over tonnage, 1888 (tons)	326,466
Grand total tonnage of Hudson River, including tonnage through State canals (tons)	18,582,596
Value of same	$485,735,094
Number of transportation companies for passengers or freight, not *including* steamboats or pleasure boats	30
Total number of passengers carried, 1889	5,000,000

In regard to the foregoing statement of the capacity of ice houses ı and near the river, as made by Mr. Weir, it may be remarked ıat Charles C. Brown, in a report on Hudson River, which appears ı the Eleventh Annual Report of the State Board of Health, gives a st of ice houses on Hudson River, with their capacity in 1889. Acording to Mr. Brown, the total capacity in that year was 2,908,000 ons, while the crop harvested frequently exceeds this quantity by 00,000 tons, which is stacked up outside and disposed of before the 'arm season begins. Mr. Weir's statistics, as stated, include the apacity of ice houses on and near Hudson River, while Mr. Brown's ıclude only those actually on the river, which probably explains the pparent discrepancy in the statistics.

STATE CANALS.

EARLY HISTORY.

The idea of a water communication between Hudson River and he West via the valley of the Mohawk had been a favored one with he statesmen of New York for many years previous to the beginning f the present century; the early projects, however, were altogether ith reference to improvement of the natural water channels and did ıot include the construction of artificial channels further than such hannels might be necessary as connecting links. Thus Sir Henry loore, governor of the colony of New York, proposed, in December, 768, an improvement of the navigation of the Mohawk River at Little 'alls by a sluice on the plan then used on the Languedoc Canal in 'rance.

In 1788 Elkhannah Watson proposed to establish a water communiation from Hudson River and Lake Ontario by way of Oneida Lake,)neida River, and Oswego River, his plan being to connect Wood 'reek with Mohawk River by a canal and to improve the Mohawk ith locks.

In January, 1791, Governor George Clinton, in an address to the egislature, urged the necessity of improving the natural water chanıels in order to facilitate communication with the frontier settlenents. Following this address, in February of the same year, a joint ommittee was appointed to inquire what obstructions in the Hudson ınd Mohawk rivers it would be proper to remove. As the result of his inquiry, an act was passed March 24, 1791, authorizing the comıissioners of the land office to explore and survey the ground from lohawk River at Fort Stanwix (now Rome) to Wood Creek with eference to constructing an artificial channel, and also to survey lohawk and Hudson rivers for improvement by locks and to estimate ıe cost of the same. A sum not exceeding $500 was appropriated to ıy the expense of such survey.

At that time the channel of commerce was by the Mohawk from lbany to Fort Stanwix in boats of about 5 tons burden. Going west

these boats carried from 1½ to 2 tons and on the easterly trip 5 tons. From Fort Stanwix there was a portage of 2 miles across the flats to Wood Creek, whence the course lay into Oneida Lake and River, and from thence into Seneca and Oswego rivers to Lake Ontario; or, from points farther west, up Seneca River to Lakes Cayuga and Seneca. At that time it cost from $75 to $100 per ton for transportation from Seneca Lake to Albany. The time occupied in going from Albany to Seneca Lake was twenty-one days, and in returning eight days.

The commissioners appointed under the act of March, 1791, were Elkhannah Watson, Gen. Phillip Schuyler, and Goldsborrow Bayner. On the 3d of January, 1792, the commissioners reported the cost of improving the route from Albany to Seneca Lake by locks and canals at $200,000, whereupon the legislature passed an act March 30, 1792, incorporating the Western Inland Lock Navigation Company, for the purpose of opening navigation by locks from Hudson River to Lakes Ontario and Seneca, and the Northern Inland Lock Navigation Company, charged with performing a like service from Hudson River to Lake Champlain. The capital stock of each company consisted of 1,000 shares of $25 each, but the companies were afterwards allowed a capital stock of $300,000 and an increase of the same from time to time. George Washington, the original promoter of canals in the United States, was the first president of the Western Inland Lock Navigation Company.

In March, 1795, an act was passed directing the State treasurer to subscribe 200 shares to these companies, of $50 each. State aid was again granted by an act passed in April, 1796, by which the Western Inland Lock Navigation Company was loaned $37,500, and a mortgage taken by the State on the company's property at Little Falls. In that year a route was opened from Schenectady to Seneca Falls for boats carrying 16 tons. The locks at Little Falls were first built of wood, then of brick, and finally of stone; the remains of the latter are said still to exist. Dividends were paid for a number of years on the stock of the company making these improvements. The tariff levied for a barrel of flour carried 100 miles was 52 cents, and for a ton of goods, $5.75.

Finally New York entered upon its era of inland-water improvements under the auspices of the State itself. On April 15, 1817,[1] an act was passed entitled "An act respecting navigable communication between the great eastern and northern lakes and the Atlantic Ocean," in which it was provided that whenever in the opinion of the canal commissioners it should be for the interests of this State that all the interest and title in law and equity of the Western Inland Lock Navigation Company shall be vested in the people of this State, it should

[1] The full authority for the construction of the Erie and Champlain canals may be found in two acts, the first being chapter 237 of the laws of 1816, passed April 17, 1816; the second being chapter 232 of the laws of 1817, passed April 15, 1817. There is more or less confusion of these two dates in early canal literature.

be lawful for the canal commissioners to pass a resolution to that effect. The act then provides a procedure for taking the property of this company after the passage of such a resolution by the canal commissioners. Following this act the rights of the company were soon transferred to the State, and the property operated for two or three years thereafter under State auspices. In 1821 the State collected the sum of $450.56 for tolls charged from Rome to the lower lock at Little Falls on account of transportation over the route formerly controlled by the Western Inland Lock Navigation Company.

Chapter 144 of the laws of 1813 incorporated the Seneca Lock Navigation Company for the purpose of constructing a canal from Cayuga Lake to Seneca Lake. The rights of this company were purchased by the State, pursuant to chapter 271 of the laws of 1825. The two companies, the Western Inland Lock Navigation Company and the Seneca Lock Navigation Company, may be considered the forerunners of Erie Canal.

About $100,000 was expended by the Northern Inland Lock Navigation Company on locks around the falls at Cohoes and for their improvement, all of which proved a total loss, the rights of the company being finally transferred to the State before navigation from Hudson River to Lake Champlain was actually opened.

The amount expended by the Western Inland Lock Navigation Company up to December, 1804, was $367,743, which was increased to $480,000 in 1813, and to a total of $560,000 before the works were finally transferred to the State. The mistake of first constructing wooden locks proved a severe loss to the company, as all the original locks at Little Falls, German Flats, and Rome rotted away in about six years. The facilities afforded by these companies were undoubtedly inadequate to the demands of the rapidly growing western section, and accordingly an active agitation finally began for some more extended means of communication.

The early work was, as we have seen, entirely in the direction of the improvement of natural channels, the extent of artificial channels for the whole route from Hudson River to Seneca Lake being only 15 miles. About 1803, however, the project for an artificial canal connecting Lake Erie with tide water in the Hudson was broached by Gouverneur Morris. In 1807 Jesse Hawley wrote a series of articles on the subject, and in 1808 the legislature directed the surveyor-general, Simeon De Witt, to make a survey of such a route. This survey was made by James Geddes, who reported on January 20, 1809. In 1810 the legislature appointed commissioners to prosecute further examinations. This commission made its first report in March, 1811. After discussing the route as proposed, from Hudson River to Lake Ontario, it recommended the inland route to Lake Erie with a direct descent from Lake Erie to Hudson River. Following this report a bill was passed by the legislature reappointing the commissioners of

the previous year, with the addition of Robert R. Livingston and Robert Fulton, and extending the powers of the commissioners and adding to the appropriation for its work. The war of 1812 came on, however, and the canal project was temporarily dropped until 1816, when De Witt Clinton presented a memorial to the legislature from the city of New York urging action toward the construction of the canal. Finally the act of April 15, 1817, was passed creating a permanent board of canal commissioners,[1] which entered at once upon its duties, and providing for the construction of artificial navigation from Lake Erie to tide water on Hudson River, and also from Lake Champlain to tide water on the Hudson. The dimensions of the proposed canals were fixed by the commissioners as follows: For Erie Canal a bottom width of 28 feet, surface width 40 feet, and depth 4 feet, with locks 90 feet long and 15 feet wide; for Champlain Canal a bottom width of 20 feet, surface width 30 feet, and depth 3 feet, with locks 75 feet long and 10 feet wide.

Ground was broken for Erie Canal at Rome, July 4, 1817, and the section from Utica to Seneca River completed October 22, 1819, a boat passing from Rome to Utica on that day. Champlain Canal was opened in part for navigation November 24, 1819. The route for Erie Canal from Seneca River west was also explored in 1819, and the final location, from Seneca River to Rochester, made in 1821. The principal engineers were James Geddes, Benjamin Wright, and Canvass White.

The annual report of the canal commissioners, dated January 31, 1818, gives details of the system adopted for the construction of the canal. They state that they had decided to complete the middle section first, 58 miles of which were put under contract during the year 1817, this portion being wholly on the summit level. The whole labor performed in 1817 was equal to the completion of 15 miles. In indication of the easy character of the work, the commissioners state that three Irishmen finished 3 rods of canal in 4 feet cutting in five and one-half days, and that on the 58 miles under contract only half a mile required puddling.

The engineer's original estimate of the cost of the middle section, completed in 1819, was $1,021,851. The actual cost was $1,125,983. This increase, as stated by the commissioners, was due to change of prism and structures.

While the State canals were in progress the Seneca Lock Navigation Company, authorized by chapter 144 of the Laws of 1813, had been engaged in constructing a canal between Seneca and Cayuga lakes, including a series of locks at Seneca Falls. On June 14, 1818, a loaded boat from Schenectady, 16 tons burden, passed the newly-constructed locks at Seneca Falls. Along Mohawk River the passage

[1] The permanent board of canal commissioners of 1817 included the following men: De Witt Clinton, president; Stephen Van Rensselaer, Samuel Young, Joseph Ellicott, and Myron H[illegible], their appointment having been first authorized by the act of April 17, 1816.

A. ERIE CANAL AT BUFFALO, NEW YORK

B. BLACK ROCK GUARD LOCK ON ERIE CANAL

boats of this size was effected through the locks of the Western and Lock Navigation Company, Erie Canal not being open for igation at that date. The locks at Seneca Falls cost $60,000. The . charged during 1818 was equivalent to 9 cents per ton per mile.

Champlain Canal was, as stated, opened for navigation November 1819, from the Hudson at Fort Edward to Lake Champlain. The imated cost of this section was $250,000, but on account of chang- ; its dimensions to the same size as Erie Canal the revised estimate ounted to $333,000. The canal was finally completed from Lake amplain to Albany on September 10, 1823.

Work on Erie Canal proceeded during the years from 1820 to 1825, the former year 94 miles being in operation and in the latter 363. was finally completed from Albany to Black Rock on October 26, 25, on which day the first boat ascended the Lockport locks and ssed through the mountain ridge into Lake Erie. Uninterrupted vigation was thus obtained from that lake to the Atlantic Ocean for ats of an average of about 40 tons burden. The event was made a la day the whole length of the canal.

The construction of Erie Canal was due to the unbounded perse- rance and genius of one man—Governor De Witt Clinton—who, en one studies the early history of Erie Canal, stands forth as the ossal figure of the enterprise.

The total expenditure on Erie and Champlain canals to January 1, 6, was $9,474,373.14, from which should be deducted for pay of gineers and commissioners, the acquisition of water rights, land nages, the construction of feeders, repairs, Black Rock Harbor, vering Onondaga outlet, Salina and Onondaga side cut, Waterford d Troy side cuts, Troy dam, and Glens Falls feeder, the sum of 621,274. Hence the actual cost of construction of the canal proper s $7,853,099, which, on the aggregate length of 433 miles, equals 3,136 per mile, or, taking into account the various extensions umerated and the engineering as necessary items of expenditure, e original cost per mile of the Erie and Champlain canals may be aced at $21,881 per mile.

Between 1825 and 1833 work was begun on a number of lateral nals—as, for instance, Oswego Canal, begun in 1826 and completed 1828; Cayuga and Seneca Canal, begun in 1827 and completed in 29; Chemung Canal, begun in 1831 and completed in 1832, and ooked Lake Canal, begun in 1831 and completed in 1833. The total st of all the canals, including interest on loans up to March 23, 1833, s $11,460,066.77.

Chenango Canal was begun in 1833. The total amount expended all the canals, including original construction, extensions, main- ance, repairs, and interest on loans, to the end of 1834 was , 798,438, and the total amount of tolls received from 1820 to 1834, lusive, was $10,000,730.97—that is to say, at the end of ten years

from the original completion of Erie Canal the amount returned to the State was nearly 78 per cent of the total cost to that date. This fact is of the greatest interest because it indicates that from the very beginning the New York State canal system was operated as a purely business enterprise. It is clear, then, that in reality the State of New York, in constructing its internal navigation system, went into the transportation business; by that statement it is meant that the State managed the affairs of the canals precisely as a private company would have managed them—that is, the State built its canal system and levied as heavy tolls as the articles transported would stand.

By way of illustrating how thoroughly the State was in the transportation business and on exactly the same basis as transportation companies, it may be cited that in 1830 the legislature sent a communication to the commissioners of the canal fund asking if it were not possible to increase the rate of toll on many of the articles transported, and stating that it seemed necessary, in order to meet all the interests involved, that the canals yield somewhat greater revenue. The commissioners replied to this communication at length, giving in detail the amount of toll levied on different articles transported, and finally concluded with the statement that it would be impossible to increase the tolls materially, because the articles transported were at that time taxed all they would stand. If the rate of toll were made materially greater, many articles would not be transported on the canals, but would go by other channels, as by St. Lawrence River and by the Great Lakes. The State would thus lose the benefit derived from carrying them.

About 1833 to 1835 railroads began to attract attention as means of transportation, and in 1835 John B. Jervis, Holmes Hutchinson, and Frederick C. Mills, who were the principal canal engineers of that day, were instructed to report on the relative cost of transportation on canals and railroads. In an introduction to their report by William C. Bouck and Michael Hoffman, canal commissioners, it is stated that it will not be difficult to show that the expense of transportation on railroads is materially greater than on canals. But in addition to this there were other important considerations in favor of canals:

(1) A canal may be compared to a common highway on which every man can be the carrier of his own property, therefore creating the most active competition, and thus reducing the expense of transportation to the lowest rates. The farmer, merchant, and manufacturer can avail themselves of the advantages of carrying their own product to market in a manner best comporting with the interest of each individual.

(2) Much of the property carried on the canals is carried by transportation companies, although the largest portion is carried by individuals and small associations. The individual who becomes the

carrier of his own product has the advantage of paying nearly one-half of all the expense of transportation in the regular course of his business, and the cash disbursements do not often much exceed the payment of the tolls. To the farmer the profits on return freight, in many instances, give a full indemnity for the expense of taking his cargo to market. On railroads, on the other hand, the proprietors must necessarily be the carriers.

A fixed popular belief in the two principles laid down by Messrs. Bouck and Hoffman in their introduction to the transportation report of 1835 has been the cause of a great deal of mistaken policy in the State of New York. For instance, nearly every year since the beginning of the railway era the newspapers of the State have teemed with the statement that the State must necessarily maintain the canal system in order to check the exorbitant tariff demands of competing railways. As we have seen, in 1830, just before the beginning of the railway era, the State was taxing every article transported upon the canals all that it would stand, and the system of excessive State tariffs was continued until a few years later, when the competition of the railways forced a reduction in the tariff for transportation on the canal.

GROWTH AND DECLINE OF CANAL TRANSPORTATION.

A number of reports bearing on transportation questions were submitted in 1835, and finally the fixed policy was adopted of enlarging Erie Canal, the act authorizing what is known as the Erie Canal enlargement being passed in that year. The law authorizing the enlargement directed the construction of double locks and a prism with a width at water surface of 70 feet, and a depth of 7 feet, the locks to be 110 feet long and 18 feet wide. It was estimated that an enlargement to this extent would save 50 per cent in cost of transportation, exclusive of tolls. The enlargement to this standard width and depth was begun in 1836 and continued to 1842, when the legislature directed the suspension of expenditures. In 1847 the work of enlargement was resumed, and substantially completed in 1862. Since that time to the work now in progress under the authority of chapter 79 of the laws of 1895 there has been no change in the standard of width at water line of 70 feet and depth of 7 feet. As an interesting fact it may be pointed out that while the enlargement authorized in 1835 led to vast increase in the transportation business on the State canals the cost of transportation gradually decreased, one chief cause of such decrease being the competition of railways, until in 1883 the competition from this source became too sharp to maintain longer transportation on the canals if any toll at all were charged. The canals were then made free by legislative enactment.

The popular notion, formerly prevalent in New York, that it has been necessary to maintain the State canals in order to regulate the railways, is seen to be far from true. The railways have regulated the canals quite as much or more than the canals have regulated the railways. Indeed, the railways must be considered as having the better of it, because the State has been obliged absolutely to do away with all tolls on the canals in order to insure their obtaining business at all.

In illustration of the value of the water resources afforded by the Great Lakes in conjunction with the New York State internal navigation system, the following statement of receipts of flour, wheat, corn, oats, barley, and rye at Buffalo for certain years, from 1836 to 1896, inclusive, is given.

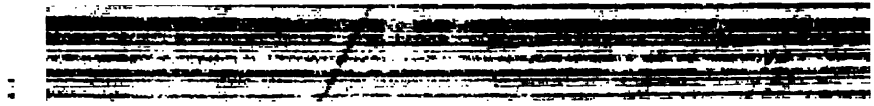
Receipts of flour, wheat, corn, oats, barley, and rye at Buffalo from 1836 to 1896.

Year.	Flour.	Wheat.	Corn.	Oats.	Barley.	Rye.
	Barrels.	*Bushels.*	*Bushels.*	*Bushels.*	*Bushels.*	*Bushels.*
1836	139,178	304,990	204,355	28,640	4,876	1,500
1840	597,142	1,004,561	71,337	.....	.....	.....
1845	746,750	1,770,740	54,200	23,300	.....	.....
1850	1,103,039	3,681,347	2,593,378	357,580	3,627	.....
1855	937,761	8,022,126	9,711,430	2,693,222	62,304	299,591
1860	1,122,335	18,502,615	386,217	1,209,594	262,158	80,822
1865	1,788,393	13,437,888	19,840,901	8,494,799	820,563	877,676
1870	1,470,391	20,556,722	9,410,128	6,846,983	1,821,154	626,154
1875	1,810,402	32,987,656	22,593,891	8,494,124	916,889	222,126
1880	1,317,911	40,510,229	62,214,417	649,351	335,925	743,451
1885	2,993,280	27,130,400	21,028,230	767,580	577,230	309,370
1890	6,245,580	14,868,630	44,136,660	13,860,780	5,165,700	1,281,050
1895	8,971,740	46,848,510	38,244,960	21,943,680	10,253,440	787,340
1896	10,384,184	54,411,207	47,811,010	40,107,499	16,697,744	4,404,354

A comparison of the statistics of railroad and canal traffic shows at once the vast preponderance of freight carried by the several railways centering at New York in comparison with that carried by Erie Canal. In spite of the fact that the canal was made free in 1883, figures indicate that since that time there has been a continual decrease in the amount of freight carried on the canals. Probably no feature of this change is more significant than that of the internal movement in New York State. In 1889 the total movement within the State was 1,438,759 tons, while in 1896 it was only 565,482 tons. These statistics show at once the decreasing estimation in which Erie Canal as a means of transportation is held by the great body of shippers in the State of New York. One of the results of this decrease is graphically

U. S. GEOLOGICAL SURVEY

EFFECT OF DECREASE OF BUSINESS ON ERIE CANAL



ed in Pl. X, showing the fleet of boats tied up and awaiting ess.

way of illustrating the growth and decline of the business of the York State canals from about 1835–36 to the present time the ing statement of total tonnage of all freight on the canals, ding and descending, and the value of the same for certain years 1837 to 1896, inclusive, is presented:

nage of freight on New York State canals and value of same, 1837 to 1896.

Year.	Tons.	Value.
1837	1,171,296	$55,809,288
1840	1,416,046	66,303,892
1845	1,977,565	100,629,859
1850	3,076,617	156,397,929
1855	4,022,617	204,390,147
1860	4,650,214	170,849,198
1865	4,729,654	256,237,104
1870	6,173,769	231,836,176
1875	4,859,958	145,008,575
1880	6,457,656	247,844,790
1885	4,731,784	119,536,189
1890	5,246,102	145,761,086
1895	3,500,314	97,453,021
1896	3,714,894	100,039,578

hout analyzing the figures in detail, it is sufficient to point out it is true, as popularly supposed, that Erie Canal ought to be ained as a medium of competition with the railways, the figures d from the annual statements of the chief competitor of Erie must be taken as conclusive that the competition has, on the been a failure. The railway, developed as a private enter- has not only been able to carry freight as cheaply as the canal, s been able to charge for the same and do the work at a profit. year ending June 30, 1897, the New York Central and Hudson Railway Company paid a dividend on its stock of $4,000,000, s carrying $51,866.80 to the surplus account, whereas the canal, gh all tolls were removed in 1883, has still been unable to com- Among the chief reasons for this result we may mention lack anization of the canal system. The perpetuation of the idea ne advantage of the canals was that they were common high- on which each man could carry his own products to market has l largely to produce this unsatisfactory result. Thus far there ver been any systematic organization for obtaining business for nal. The boats are owned by small proprietors, each operating ne to three or four boats. When cargoes in hand are discharged

at either terminus, each owner solicits another cargo. The results are delays, half cargoes, and consequent loss. During the last few years it has been only by the most rigid economy that Erie Canal boatmen could live. On the other hand, the business of soliciting freight for the railways is compactly organized and every possible advantage taken of the situation.

However unsatisfactory it may seem to the individual boatman, the future of effective transportation on Erie Canal depends, in the opinion of the author, on the organization of large transportation companies which conduct the business of carrying freight by canal on the same business basis as adopted by railways. As to the equity of the State furnishing and maintaining a waterway on which transportation may be conducted by such corporations at a profit the author expresses no opinion further than to point out that the official discussion of such a proposition by the State engineer and surveyor in his annual report for the year ending September 30, 1896, may be taken to indicate that the day of Erie Canal as a State waterway has passed.

COST AND REVENUES OF THE NEW YORK STATE CANAL SYSTEM.

The accompanying table exhibits the total cost of construction, maintenance and operation, and the total revenues from all sources of the several canals of New York from their original inception to September 30, 1892.

Cost of construction, maintenance and operation, and revenues of New York canal system.

Canals.	Cost of construction.	Cost of maintenance and operation.	Total cost.	Revenue from all sources.	Loss.	Gain.
Erie and Champlain	$57,688,676	$41,582,759	$99,271,435	$128,191,068	.....	$28,919,633
Oswego	4,643,921	3,736,676	8,380,597	3,715,567	$4,665,030	.....
Cayuga and Seneca	1,886,662	1,157,754	3,044,416	1,055,016	1,989,400	.....
Black River	4,077,882	2,082,251	6,160,133	305,663	5,854,470	.....
Oneida River improvement	233,962	41,236	275,198	214,428	60,770	.....
Oneida Lake	513,439	144,060	657,499	65,180	592,319	.....
Baldwinsville (so called)	31,000	18,039	49,039	1,261	47,778	.....
Seneca River towing path	1,602	20	1,622	7,782	.....	6,160
Cayuga Inlet	2,020	948	2,968	7,534	.....	4,566
Crooked Lake (abandoned)	395,092	424,658	819,750	45,490	774,260	.....
Chenango (abandoned)	4,807,952	2,105,217	6,913,169	744,027	6,169,142	.....
Chemung (abandoned)	1,512,041	2,022,259	3,534,300	525,565	3,008,735	.....
Genesee Valley (abandoned)	6,741,839	2,814,809	9,556,648	860,165	8,696,483	.....
Total	82,536,088	56,130,686	138,666,774	135,738,746	31,858,387	28,930,359

The total cost of Crooked Lake, Chenango, Chemung, and Genesee Valley canals, which were abandoned under the provisions of the law of 1877, was $20,823,867, and the total revenue from all sources $2,175,247. The total loss on the abandoned canals was, therefore, $18,648,620. The following is the complete financial exhibit of all canals from their inception to September 30, 1892: Total loss on abandoned canals, $18,648,620; net gain on canals now in operation, $15,720,592; loss on the whole system, $2,928,028.

IMPROVEMENT OF ERIE CANAL.

The canal improvement now in progress was formulated by State Engineer and Surveyor Horatio Seymour, jr., about 1878 or 1879. In his annual report for the fiscal year ending September 30, 1878, Mr. Seymour discusses extensively transportation questions as related to Erie Canal, pointing out that transportation can be cheapened in two ways—by increasing the tonnage of boats or by increasing their speed. As to increasing the tonnage of boats, he states that two methods may be used—the locks may be lengthened or the depth of water may be increased. He refers to experiments made by Elnathan Sweet by which it was shown that the best form of waterway should have a cross section 5.39 times the immerged section of the boat, and a surface width 4.5 times the width of the boat. The width of the canal, Mr. Seymour states, is substantially what it should be, but it lacks the necessary depth in order to conform to the law of relation of cross section of water to immerged section of boat, as determined by Mr. Sweet. Reports are submitted showing that an additional depth of 1 foot could be obtained for about $1,100,000.

The matter of making the improvement, however, remained in abeyance until the passage of an act in 1895, which provided for submitting to the people at the State election in November of that year the question as to whether an improvement by deepening 2 feet should be undertaken, at an expense of $9,000,000. Section 3 of chapter 79 of the laws of 1895 reads as follows:

Within three months after issuing of the said bonds the superintendent of public works is hereby directed to proceed to enlarge and improve the Erie Canal, the Champlain Canal, and the Oswego Canal; the said improvement to the Erie and Oswego canals shall consist of deepening the same to a depth of not less than 9 feet of water, except over and across aqueducts, miter sills, culverts, and other permanent structures, where the depth of water shall be at least 8 feet, but the deepening may be performed by raising the banks wherever the same may be practicable; also the lengthening or improving of the locks which now remain to be lengthened, and providing the necessary machinery for drawing boats into the improved locks, and for building vertical stone walls, where, in the opinion of the State engineer and surveyor and superintendent of public works, it may be necessary. The improvement upon the Champlain Canal shall consist in deepening the said canal to 7 feet of water, and the building of such vertical stone walls

as, in the opinion of the State engineer and surveyor and superintendent of public works, may be necessary.

The necessary preliminary work was so far completed that bids for constructing the improvement were called for in October, 1896, and shortly thereafter contracts for work amounting to about $4,000,000 were awarded. The canal was closed December 1, 1896, and soon after work was begun and continued until May 5, 1897, when navigation was opened for the season of 1897, and all contractors not working with dredges discontinued operations until the winter of 1897-98. The act authorizing the present improvement provided, as we have seen, that the deepening of the canal prism might be accomplished either by excavation or by raising the side walls. As a matter of fact, sometimes one method and sometimes the other has been followed, depending upon the conditions of each level. Usually, however, a part of the increased depth has been obtained by raising the side walls, with the balance secured by excavation.

As is indicated in the extract from the act authorizing the present improvement, the work includes the lengthening of such locks as have not been previously lengthened. The original locks of the enlargement of 1836 to 1862 were 110 feet long by 18 feet wide. In 1885 work was begun lengthening these locks to 220 feet, or about 210 feet in the clear, thus permitting two boats to pass through at one lockage. Up to the present time 42 of the 72 original locks have been lengthened. The 30 locks yet remaining to be dealt with are mostly bunched in groups or flights, as, for instance, at Cohoes, where 16 locks effect a change of level of 140 feet, and at Lockport, where 5 locks effect a change of 58 feet. In order to lengthen locks built in flights it would be necessary to entirely reconstruct them, and as the restricted space, especially at Lockport, would render this a very difficult thing to do in one winter season, it has therefore been proposed to construct at Cohoes and Lockport, and possibly at Newark, vertical lift locks to take the place of the ordinary locks now in use at these places. The State engineer and surveyor has completed the plans for the proposed lift lock at Lockport, which is so located as not to interfere with the use of the locks now in place there during its construction. It has been announced that the $9,000,000 appropriated will fall about $7,000,000 short of completing the work of deepening and lengthening on the lines thus far carried out, and in consequence the matter of building the lift locks is held in abeyance.[1]

---

[1] For engineering and other details of the canal improvement now in progress see Eng. News, Vol. XXXVIII (Sept. 2, 16, and 23, 1897). See also Effect of depth upon artificial waterways, by Thomas C. Clark: Trans. Am. Soc. Civil Eng., Vol. XXXV, pp. 1-40. Also Eng. News, January 6, 1898, for discussion of the question, What shall New York do with its canals?

DESCRIPTION OF THE CANALS NOW IN OPERATION, AND THEIR WATER SUPPLY.

Following are some of the main facts in regard to the principal canals—Erie, Champlain, Oswego, and Black River—now in operation in the State of New York. Similar facts for Oneida Lake Canal, Oneida River improvement, the Cayuga and Seneca Canal, and others, may be obtained by reference to the annual reports of the superintendent of public works.

Length, capacity, and cost of New York State canals.

ERIE CANAL.

	Original canal.	Enlargement.
Length, in miles	363.00	351.78
Lockage, in feet	675.50	645.80
Average burden of boats, in tons	70.00	210.00
Maximum burden of boats, in tons	76.00	240.00
Construction authorized	Apr. 15, 1817	May 11, 1835
Construction completed	Oct., 1836	Sept., 1862
Actual cost of construction	$7,143,789	$44,465,414

CHAMPLAIN CANAL.

Length of canal, in miles	66
Length of feeder, in miles	7
Length of pond, in miles	5
Total, in miles	78
Construction authorized	Apr. 15, 1817
Glens Falls feeder authorized	Apr., 1822
Estimated cost of canal	$871,000
Total cost of canal and feeder to 1868	2,378,910
Total cost, including improvements and enlargements, to 1875	4,044,000

OSWEGO CANAL.

	Original canal.	Enlarged canal.
Length, in miles	38.00	38.00
Lockage, in feet	154.85	154.85
Average burden of boats, in tons	62.00	225.00
Construction authorized	Apr., 1825	Apr., 1854
Construction completed	Dec., 1828	Sept., 1862
Actual cost of construction	$565,473	$4,427,589

Length, capacity, and cost of New York State canals—Continued.

BLACK RIVER CANAL.

Length of canal, Rome to Lyons Falls, in miles	35.00
Length of improved river to Carthage, in miles	42.00
Length of navigable feeder, in miles	10.50
Lockage, in feet	1,082.25
Average burden of boats, in tons	48.00
Construction authorized	Apr., 1836
Construction completed	1849
Actual cost of construction	$3,581,954

EASTERN DIVISION OF ERIE CANAL.

Erie Canal is divided into three divisions, known as the eastern, the middle, and the western. The eastern division embraces the portion of the canal, with its feeders and side cuts, extending from the Hudson River at Albany to the dividing line between the counties of Herkimer and Oneida, and the whole of the Champlain Canal, with its feeders, ponds, and side cuts. The entire mileage of canals, feeders, and river improvements on the eastern division is as follows:

Mileage of eastern division of Erie Canal.

	Miles.
Erie Canal, Albany to east line of Oneida County	103.24
Fort Schuyler and West Troy side cuts	0.35
Albany basin	0.77
Champlain Canal, including Waterford side cut	66.00
Navigable river above Troy dam	3.00
Glens Falls feeder	7.00
Navigable river above Glens Falls feeder dam	5.00
Total	188.36

Mileage of unnavigable feeders of the eastern division of Erie Canal.

	Miles.
Mohawk River at Rexford Flats	0.39
Mohawk River at Rocky Rift	3.92
Mohawk River at Little Falls	0.19
Schoharie Creek	0.64
Total	5.13

WATER SUPPLY OF THE EASTERN DIVISION.

To the west of Little Falls, on the Erie Canal, lies 19.2 miles of the eastern division, supplied from the reservoirs and streams of the middle division. East of Little Falls the supply is from Mohawk River, through Little Falls, Rocky Rift, and Rexford Flats feeders, and from Schoharie Creek through Schoharie Creek feeder. As to the quantity of water used on that portion of Erie Canal included in the eastern division very little is known. With the exception of a few thousand cubic feet per minute received from the middle division, the supply is, as just indicated, all derived from Mohawk River and

ERIE CANAL AT LITTLE FALLS, NEW YORK.

tributary, Schoharie Creek. Thus far no measurements of the ual quantity used have been made. Probably the total diversion ounts in dry weather to from 500 to 600 cubic feet per second. ne of this is returned to Mohawk River by leakage and wastage, just what proportion is returned, and what finally delivered either Hudson River at Albany or by the Troy and Fort Schuyler side s, is not known. In view of the magnitude of the power develop- nt on Mohawk River at Cohoes it appears very desirable that such etermination be made.

'he water supply of Champlain Canal is derived from Wood Creek l several small streams to the north of Fort Edward, Glens Falls der, Hudson River feeder, from Hudson River itself at Saratoga n, and from Mohawk River at the Cohoes dam. An investigation he amount of water diverted from Hudson River for the supply of amplain Canal was made by the author in the fall of 1895.

s already stated, Champlain Canal is fed from Hudson River by ns Falls feeder, which connects with the river about 2 miles above ns Falls and from the Saratoga dam at Northumberland.

he length of Glens Falls feeder, from the guard lock at its head to re it enters Champlain Canal, about 2 miles above Fort Edward, is miles. From this point the water in the canal flows both north south, the total length of canal fed by Glens Falls feeder being 1 miles. Fort Edward level, into which Glens Falls feeder delivers er, is a summit level, and hence the water delivered into it, less losses by percolation, evaporation, etc., is partly discharged into e Champlain and partly into Hudson River at Saratoga dam. mplain Canal crosses through the pond formed by Saratoga dam n the east side to the west of the Hudson and again passes out of river, taking a full supply therefrom at the village of Northum- land, from which point to Mohawk River at Cohoes the distance is 6 miles. The water from this section by passing into the Mohawk lly reaches the Hudson above the Troy dam. The canal crosses awk River at Cohoes, taking water therefrom to supply the section n Cohoes to near West Troy, a distance of 2.36 miles. A small ount of water also passes from Champlain Canal to the Hudson ough the Waterford side cut.

ince the construction of Glens Falls feeder there have existed ous leaks though the seamy limestone rock in which the feeder is avated at and below the village of Glens Falls. It is claimed that losses through these seams have generally increased, until for sev- years past they have amounted to about 50 per cent of the total into the feeder at the guard lock.

his leakage has been repeatedly complained of by the owners of er power at Glens Falls and several attempts to check it have been le, but without much avail. The river falls 38 feet at Glens Falls, the owners of the water power there claim that this leakage,

Mileage of navigable feeders of the middle division of Erie Canal.

	Miles.
Limestone Creek feeder	0.83
Butternut Creek feeder	1.67
Camillus feeder	1.04
Delta feeder	1.40
Black River feeder	11.29
Total	16.23

The total length of canal, river improvement, and navigable feeders on the middle division is thus found to be 299.08 miles. The following feeders of the middle division are not navigable:

Mileage of unnavigable feeders of the middle division of Erie Canal.

	Miles.
Chenango Canal, summit level	5.31
Leland Pond feeder	.31
Madison Brook feeder	2.99
West Branch feeder	5.83
Bradleys Brook feeder	.67
Hatch Lake feeder	.23
Kings Brook feeder	1.87
Oriskany Creek feeder	.53
Mohawk feeder at Rome	.01
Oneida Creek feeder	2.91
Cowasselon Creek feeder	.30
Chittenango Creek feeder	.25
Cazenovia Lake outlet	.51
Tioughnioga River feeder	1.00
De Ruyter reservoir outlet	.12
Orville reeder	.55
Camillus feeder (unnavigable portion)	.65
Carpenter Brook feeder	.18
Skaneateles Creek feeder	.09
Putnam Brook feeder	.20
Centerport feeder	.18
Owasco Creek feeder	2.10
Lansing Kill feeder	1.80
Sugar River feeder	.14
Canachagala Lake outlet	.16
Total	29.04

Rome level, which is a summit level, extends from lock No. 46 to lock No. 47, a distance of 55.96 miles. The following is the estimated water supply of this level before the beginning of the enlargement now in progress.

ERIE CANAL AT SYRACUSE, NEW YORK.

Water supply of Rome level, Erie Canal.

	Second-feet.
Lelands Pond, Madison Brook reservoir, Eaton Brook reservoir, Bradley Brook reservoir, Hatch Lake, Kingsley Brook reservoir, and Oriskany Creek feed through the Chenango Canal, Oriskany Creek, and Oriskany Creek feeder into the Rome level, 6 miles west of lock No. 46	100
Mohawk River, Black River, Forestport Pond, Forestport reservoir, White Lake reservoir, Chub Lake, Sand Lake, First, Second, and Third Bisby lakes, Woodhull reservoir, Twin Lakes, South Branch reservoir, North Branch reservoir, and Canachagala Lake feed through the Rome feeder and Black River Canal into the Rome level at Rome, 14 miles west of lock No. 46	217
Oneida Creek enters canal through feeder 30 miles west of lock No. 46	17
Cowasselon Creek enters canal through feeder 31.5 miles west of lock No. 46	3
Cazenovia Lake reservoir, Erieville reservoir, and Chittenango Creek enter canal through Chittenango Creek feeder, 41.5 miles west of lock No. 46; average for navigation season about	47
De Ruyter reservoir enters canal through Limestone Creek (Fayetteville) feeder, 50 miles west of lock No. 46; average for the navigation season about	32
Limestone Creek (natural flow) also enters canal through Limestone Creek (Fayetteville) feeder, 50 miles west of lock No. 46	8
Jamesville reservoir enters canal through Orville feeder, 52 miles west of look No. 46; average for navigation season	11
Butternut Creek (natural flow) enters canal through Orville feeder, 52 miles west of lock No. 46	8
Total	443

Jordan level, which is also a summit level, extends from lock No. 50 to lock No. 51 and is 14.903 miles in length. The following feeders are tributary to this level:

Water supply of Jordan level, Erie Canal.

	Second-feet.
Otisco Lake reservoir fed through Camillus feeder into the canal, 4 miles west of lock No. 50	86
Ninemile Creek (natural flow) also fed into canal through Camillus feeder	13
Carpenter Brook feeder	3
Skaneateles feeder	146
Total	248

The following feeders deliver water into the Port Byron level, which extends from lock No. 51 to No. 52, a distance of 7.79 miles:

Water supply of Port Byron level, Erie Canal.

	Second-feet.
Putnam Brook feeder at Weedsport	3
Owasco feeder	69
Total	72

Oswego Canal receives about 167 cubic feet per second from Erie Canal at Syracuse. The balance of its water supply is derived from Seneca and Oneida rivers. The total amounts to about 1,400 cubic feet per second. Seneca and Cayuga Canal receives about 67 cubic feet per second from Erie Canal at Montezuma, and 300 cubic feet

Mileage of navigable feeders of the middle division of Erie Canal.

	Miles.
Limestone Creek feeder	0.83
Butternut Creek feeder	1.67
Camillus feeder	1.04
Delta feeder	1.40
Black River feeder	11.29
Total	16.23

The total length of canal, river improvement, and navigable feeders on the middle division is thus found to be 299.08 miles. The following feeders of the middle division are not navigable:

Mileage of unnavigable feeders of the middle division of Erie Canal.

	Miles.
Chenango Canal, summit level	5.31
Leland Pond feeder	.31
Madison Brook feeder	2.99
West Branch feeder	5.83
Bradleys Brook feeder	.67
Hatch Lake feeder	.34
Kings Brook feeder	1.87
Oriskany Creek feeder	.53
Mohawk feeder at Rome	.03
Oneida Creek feeder	2.91
Cowasselon Creek feeder	.40
Chittenango Creek feeder	.28
Cazenovia Lake outlet	.51
Tioughnioga River feeder	1.40
De Ruyter reservoir outlet	.12
Orville feeder	.55
Camillus feeder (unnavigable portion)	.65
Carpenter Brook feeder	.18
Skaneateles Creek feeder	.9
Putnam Brook feeder	.30
Centerport feeder	.18
Owasco Creek feeder	2.10
Lansing Kill feeder	1.80
Sugar River feeder	.14
Canachagala Lake outlet	.16
Total	29.04

Rome level, which is a summit level, extends from lock No. 46 to lock No. 47, a distance of 55.96 miles. The following is the estimated water supply of this level before the beginning of the enlargement now in progress.

ERIE CANAL AT SYRACUSE, NEW YORK.

per second from Seneca Lake, making a total of 367 cubic feet per second.

The approximate water supply of the middle division of Erie Canal at present may therefore be summarized as follows:

Water supply of the middle division of Erie Canal.

	Second-feet.
Frankfort and Rome level	443
Jordan level	249
Port Byron level	70
Total	762
Oswego Canal:	
Supply from Seneca River	900
From Oneida River	333
Total	1,233
Seneca and Cayuga Canal	300
Grand total	2,295

The present total water supply of middle division may thus be placed, approximately, at 2,295 second-feet.[1]

RESERVOIRS OF THE MIDDLE DIVISION.

As indicated in the preceding statement of the water supply of the middle division, this division has an extensive system of reservoirs, as, for instance, Owasco Lake, the original surface of which has been raised by a dam on the outlet about half a mile below the foot of the lake. Skaneateles and Otisco lakes have also been raised by similar constructions. Proceeding east, the next reservoir is the Jamesville, formed by a dam on Butternut Creek. De Ruyter reservoir is on the dividing ridge, at the extreme head waters of Limestone Creek, and supplied by a feeder from the head waters of Tioughnioga River, a tributary of Chenango River. On Chittenango Creek, the next stream east of Limestone Creek, we find Cazenovia Lake, controlled by a dam at its foot and Erieville reservoir on the extreme head waters.

On the head waters of Oriskany Creek and Chenango River there is an extensive reservoir system, originally constructed for the purpose of supplying the summit level of Chenango Canal, but which has been retained in use for the water supply of the middle division, the water therefrom being conducted to Erie Canal at Utica, through the old prism of Chenango Canal, retained as a feeder, or to Erie Canal at Oriskany, through the natural channel of Oriskany Creek and a short feeder at Oriskany village. The reservoirs on the head waters of Oriskany Creek are Madison Brook and Lelands Pond reservoirs: those on the head waters of Chenango Creek, and which are connected with the summit level of the old Chenango Canal by a feeder

[1] This is a very general statement, and to be taken in connection with a large amount of detailed information in the reports not specifically cited at this time.

Water supply of Rome level, Erie Canal.

	Second-feet.
Lelands Pond, Madison Brook reservoir, Eaton Brook reservoir, Bradley Brook reservoir, Hatch Lake, Kingsley Brook reservoir, and Oriskany Creek feed through the Chenango Canal, Oriskany Creek, and Oriskany Creek feeder into the Rome level, 6 miles west of lock No. 46	100
Mohawk River, Black River, Forestport Pond, Forestport reservoir, White Lake reservoir, Chub Lake, Sand Lake, First, Second, and Third Bisby lakes, Woodhull reservoir, Twin Lakes, South Branch reservoir, North Branch reservoir, and Canachagala Lake feed through the Rome feeder and Black River Canal into the Rome level at Rome, 14 miles west of lock No. 46	217
Oneida Creek enters canal through feeder 30 miles west of lock No. 46	17
Cowasselon Creek enters canal through feeder 31.5 miles west of lock No. 46	3
Cazenovia Lake reservoir, Erieville reservoir, and Chittenango Creek enter canal through Chittenango Creek feeder, 41.5 miles west of lock No. 46; average for navigation season about	47
De Ruyter reservoir enters canal through Limestone Creek (Fayetteville) feeder, 50 miles west of lock No. 46; average for the navigation season about	32
Limestone Creek (natural flow) also enters canal through Limestone Creek (Fayetteville) feeder, 50 miles west of lock No. 46	8
Jamesville reservoir enters canal through Orville feeder, 52 miles west of look No. 46; average for navigation season	11
Butternut Creek (natural flow) enters canal through Orville feeder, 52 miles west of lock No. 46	8
Total	443

Jordan level, which is also a summit level, extends from lock No. 50 to lock No. 51 and is 14.903 miles in length. The following feeders are tributary to this level:

Water supply of Jordan level, Erie Canal.

	Second-feet.
Otisco Lake reservoir fed through Camillus feeder into the canal, 4 miles west of lock No. 50	86
Ninemile Creek (natural flow) also fed into canal through Camillus feeder	13
Carpenter Brook feeder	3
Skaneateles feeder	146
Total	248

The following feeders deliver water into the Port Byron level, which extends from lock No. 51 to No. 52, a distance of 7.79 miles:

Water supply of Port Byron level, Erie Canal.

	Second-feet.
Putnam Brook feeder at Weedsport	3
Owasco feeder	69
Total	72

Oswego Canal receives about 167 cubic feet per second from Erie Canal at Syracuse. The balance of its water supply is derived from Seneca and Oneida rivers. The total amounts to about 1,400 cubic feet per second. Seneca and Cayuga Canal receives about 67 cubic feet per second from Erie Canal at Montezuma, and 300 cubic feet

Reservoirs for the water supply of Erie Canal.

Reservoir.	Date of completion.	Elevation.	Distance to canal.	Flow-line area.	Height of flow line.	Average area.	Average depth.	Capacity.	Original cost.	Cost per million cubic feet.
		Feet.	*Miles.*	*Acres.*	*Ft.*	*Acres.*	*Feet.*	*Cubic feet.*		
ROME LEVEL—ERIE CANAL	.....	428	.....	.....	.....	.....	.....	.....	.....	.....
Woodhull	1859	1,854	25.0	1,236	16	1,118	18.0	876,550,000	$24,089	$27.47
Bisby Lake, No. 1 *a*	1881	2,018	27.8	156	4					
Bisby Lake, No. 2	1881	2,018	27.0	204	4	.....	3.5	40,000,000	.....	.....
Bisby Lake, No. 3	1881	2,000	26.0	41	4					
Sand Lake	1872	1,808	24.0	344	18	306	15.0	199,879,822	5,505	27.71
Chubb Lake	1881	1,599	20.5	530	22	200	4.0	34,848,000	8,134	233.43
White Lake	1881	.....	18.0	332	5	296	5.0	64,468,800	5,102	79.27
Canachagala Lake	1881	.....	32.0	347	4	320	4.0	55,756,800	1,456	26.11
North Branch Lake	1857	1,821	25.5	423	28	277	28.0	337,851,360	34,637	102.2
South Branch Lake	1859	2,019	26.0	518	26	372	26.0	421,312,320	20,168	47.87
Twin Lakes	1881	.....	22.0	212	6	175	8.0	60,984,000	.....	.....
Forestport Pond	1853	1,124	10.5	160	.....	.....	2.0	13,921,920	.....	.....
Forestport reservoir	1894	1,139	12.5	793	21	700	7.0	212,444,000	.....	.....
BOONVILLE LEVEL—BLACK RIVER CANAL	.....	1,121	25.0	.....	.....	.....	.....	.....	.....	.....
JORDAN LEVEL—ERIE CANAL	.....	408	.....	.....	.....	.....	.....	.....	.....	.....
Skaneateles Lake	1844	864	9.0	8,320	.....	.....	9.0	2,174,512,000	14,928	6.87
Otisco Lake	1869	.....	12.0	2,200	.....	.....	10.0	784,000,000	43,753	55.81
PORT BYRON LEVEL—ERIE CANAL	.....	403	.....	.....	.....	.....	.....	.....	.....	.....
Owasco Lake	1866	704	12.0	680	.....	.....	5.0	1,481,040,000	36,296	25.79
ROCHESTER LEVEL—ERIE CANAL	.....	508	.....	.....	.....	.....	.....	.....	.....	.....
CUBA LEVEL—GENESEE VALLEY CANAL	.....	1,489	.....	.....	.....	.....	.....	.....	.....	.....
Oil Creek	1858	.....	91.0	605	65	525	15.0	435,738,000	69,216	158.85
Rockville	1842	.....	8.30	.....	.....	72	20.0	18,200,000	7,711	423.68
ROME LEVEL—ERIE CANAL	.....	428	.....	.....	.....	.....	.....	.....	.....	.....
Erieville	1850	1,577	20.0	340	46	.....	21.5	318,424,000	36,857	115.68
Cazenovia Lake	1857	1,176	10.0	1,778	.....	.....	4.5	348,523,000	10,885	31.23
De Ruyter	1863	1,276	25.0	626	.....	.....	18.5	504,468,000	78,761	156.13
Jamesville	1874	585	6.0	252	.....	240	16.0	170,000,000	150,000	882.35
Hatch Lake *b*	1836	.....	36.0	134	15	.....	10.0	58,370,400	4,464	76.47
Eaton Brook *b*	1836	.....	38.0	254	60	244	50.0	553,212,000	28,059	50.72
Bradley Brook *b*	1836	.....	35.0	134	30	.....	25.0	145,926,000	16,159	110.73
Leland Pond *b*	1836	.....	25.0	173	13	150	8.0	59,287,040	8,894	150.02
Madison Brook	1836	1,127	29.0	345	55	255	45.0	460,617,000	36,501	78.80
Kingsley Brook	1867	.....	33.0	113	.....	.....	20.0	98,445,600	80,481	817.54

a Supply to canal through Forestport Pond and Black River Canal.
b Supply to canal through Oriskany Creek feeder.

SHIP-CANAL PROJECTS AND WATER SUPPLY.

By virtue of the geographic position of New York, with the Great Lakes on its western boundary and the Atlantic Ocean on its eastern, and with the commercial capital of the Western Continent as its *chief* city, all discussions of deep waterway projects from the Upper

Great Lakes to the seaboard are, necessarily, chiefly discussions of the water resources of New York. It is proper, therefore, that the several deep water projects now under discussion should be briefly noticed in a report of this character.

In February, 1895, Congress by a joint resolution authorized a preliminary inquiry concerning deep waterways between the Great Lakes and the ocean, and provided that the President should appoint three commissioners to make such inquiry. The President, under this resolution, appointed Prof. James B. Angell, of Ann Harbor, Michigan; John E. Russell, of Leicester, Massachusetts, and Lyman E. Cooley, of Chicago, Illinois. The report of the commission, as published in 1897, includes a large amount of valuable information in regard to a deep waterway from the Upper Great Lakes to the Atlantic seaboard. In regard to the State of New York, it has been pointed out by Mr. Cooley that nature has indicated two feasible routes for such a canal. The first of these is the Oswego-Mohawk-Hudson route, extending from Oswego through the valley of Oswego and Oneida rivers, and thence across the divide to the Mohawk, thence through Mohawk Valley to a point on the Hudson in the vicinity of Troy, and so on through Hudson River to tide water at New York. One objection to this route is the lockage over the summit between Lake Ontario and Mohawk Valley. Another objection is the absorption of a large quantity of water in central New York for the supply of the summit level of the canal, and which probably can be more effectively used in manufacturing; that is to say, the State of New York, by developing its manufacturing resources to their fullest extent, can realize more return from manufacturing than from the use of its inland waters for purposes of internal navigation of any kind whatever. It may be pointed out in passing that the Oswego-Mohawk-Hudson route would utilize the great natural highway which has been an easy passage to commerce from the early days of settlement on the Atlantic coast.

The second natural route through the State of New York is by way of St. Lawrence River to the head of Coteau Rapids, where the low-water level of Lake St. Francis is 153.5 feet above tide, or 68.5 feet above the low-water level of Lake Champlain. On this plan a canal would be constructed from Coteau Landing to the head of Lake Champlain, near Rouses Point, this section requiring cutting through a summit about 50 feet in height. Lake Champlain would then be utilized to Whitehall, from which point a canal would be cut through the valley leading from Whitehall to Hudson River at Fort Edward, the elevation of the water surface of the Hudson a few miles below Fort Edward being somewhat less than the low-water elevation of Lake Champlain. After reaching the Hudson the work would include the deepening of that stream to deep water, a few miles below Albany. Either of the foregoing projects would further include the construction of a ship canal connecting Lakes Erie and Ontario.

The advantage of the St. Lawrence-Champlain-Hudson over the Oswego-Mohawk-Hudson route is that the lockage would be all in one direction; that is, eastward-bound vessels would lock down all the way from Lake Erie to New York. Its disadvantages are increased length and the location of the canal connecting St. Lawrence River with Lake Champlain in Canadian territory. In regard to increased length, it is claimed that not much more time would be required in traversing it than would be consumed in locking over the Oswego-Mohawk summit.

As to the capacity of the proposed canal, the Deep Waterways Commission points out in its report that such a canal, if built, should be so carried out as to be adequate for vessels of the most economical type, not only for coasting or domestic trade but also for the foreign movement, so that commerce may be carried on directly between lake ports and other domestic and foreign ports without transshipment. Taking into account various other conditions, the commission believes that the requirements of the present demand a limiting draft in the proposed canal of 27 or 28 feet; hence, in concluding the general discussion, the commission recommends the securing of a channel of a navigable depth of not less than 28 feet.

The commission also says that, starting from the heads of Lakes Michigan and Superior, the most eligible route for a deep waterway is through the several Great Lakes and their intermediate channels and the proposed Niagara ship canal to Lake Ontario, and that the Canadian seaboard may then be reached from Lake Ontario by the way of St. Lawrence River, and the American seaboard reached from Lake Ontario by way of either the Oswego-Mohawk-Hudson route or the St. Lawrence-Champlain-Hudson route. The Deep Waterways Commission was not authorized to make any considerable expenditure for surveys, and hence the conclusions announced are to some degree tentative. In view of the uncertainty as to final cost, it is recommended that the alternative routes from Lake Ontario to the Hudson be subject to complete survey in order to obtain a full development of the governing econom[illegible]siderations, as well as to determine their relative availability.

The commission als[illegible]nds a moderate contro[illegible] Lake Erie and of Niag[illegible]above Tonawanda b[illegible] the practical details [illegible]ed in the absen[illegible] standing of the physic[illegible]ns.

The river and harb[illegible]une 3, 1896 [illegible] War to cause to be m[illegible] examin[illegible] cost of constructing a [illegible] the m[illegible] within the United S[illegible] waters of Hudson Riv[illegible] *nage* of the lakes to th[illegible]

was submitted by Maj. Thomas W. Symons, of the Corps of Engineers, dated June 23, 1897.[1]

Major Symons states that there are three possible routes for the ship canal, entirely within the territory of the United States, from the Great Lakes to the navigable waters of the Hudson, as follows:

(1) From Lake Erie via the upper Niagara River to the vicinity of Tonawanda or La Salle; thence by canal, with locks, either to the lower Niagara at or near Lewiston, or to some point on Lake Ontario; thence through Lake Ontario to Oswego; thence up Oswego and Oneida rivers to Oneida Lake, and through Oneida Lake; thence across the divide to Mohawk River, and down that river to the Hudson at Troy; thence down the Hudson. This he designates as the Oswego route. From Oswego to Hudson River it is, in effect, the Oswego-Mohawk-Hudson route, already described.

(2) To follow either the line of Erie Canal from Lake Erie to the Hudson, or this line so modified as to provide for a continuously descending canal from Lake Erie to the Hudson. This he designates as the Erie Canal route.

(3) This route coincides with the first from Lake Erie to Lake Ontario, but runs thence through Lake Ontario to St. Lawrence River and down said river to some point near Ogdensburg; it then crosses the State of New York to Lake Champlain and up that lake to Whitehall; and thence follows in general the route of the Champlain Canal to Hudson River at Troy. This route, however, Major Symons pronounces impracticable.

There is also discussed a fourth route—the St. Lawrence-Champlain—all of which, except a small portion, is within the United States. This route would be via Niagara Falls, Lake Ontario, the St. Lawrence, Caughnawaga, and Richelieu rivers, Lake Champlain, and the Hudson.

The opinion is expressed that the best route for the contemplated ship canal is that via Niagara River, Lake Ontario, Oswego and Oneida rivers, Oneida Lake, and Mohawk and Hudson rivers, and that to build such a canal by any of the possible routes mentioned would, at a rough estimate, cost $200,000,000, the exact figure depending very [illegible]ly upon the action of the State of New York in regard to the State [illegible], feeders, reservoirs, etc.; and that to maintain the canal and to [illegible] in repair, in[illegible]ng the maintenance of river channels, reser- [illegible] at a rough esti[illegible],000,000 a year. [illegible] ship canal wo[illegible]o special mili- [illegible]ction is not w[illegible]ng undertaken [illegible] because th[illegible] benefits to be [illegible]mmensur[illegible] cost. [illegible] the o[illegible]rie Canal, when [illegible]ns of [illegible] New York, may

[1] [illegible] June 30, 1897.

give, if State restrictions are removed, commercial advantages practically equal to those to be derived from the proposed ship canal, and that if Erie Canal be further improved by enlargement to a size sufficient for 1,500-ton barges, making such alterations in alignment as to give a continuously descending canal all the way from Lake Erie to the Hudson, and canalizing Mohawk River, the improved canal, navigated by barges, would render practicable the transportation of freight between the East and the West at a lower rate than by a ship canal navigated by large lake or ocean vessels. The difficulty of navigating large vessels through long, shallow canals is the loss of time and the consequent great increase in the pro rata expense account, as compared with the actual amount transported between termini. Major Symons is also of opinion that the enlargement of Erie Canal on these lines is a project worthy of being undertaken by the General Government, because the benefits to be derived would be commensurate with the cost, which he estimates at approximately one-fourth that of a ship canal, or $50,000,000.

Space will not permit abstracting in detail these several interesting reports on ship canals across the State of New York. To understand the subject in all its bearings, reference must be made to the original reports.

The Oswego-Mohawk-Hudson route is discussed in a report by Albert J. Himes in the Report of the State Engineer and Surveyor for 1895.[1]

In this report Mr. Himes expresses the opinion that a sufficient water supply could not be obtained for a high summit level across the divide, and hence the canal must be cut from the level of Oneida Lake through to the corresponding level in Mohawk Valley. In this way he proposes to use Oneida Lake as a storage reservoir from which to discharge water both ways to Oswego and Mohawk rivers. By this plan the surface of Oneida Lake would be raised 10 feet, furnishing 1,100 second-feet continuously for seven months. Without going into detail, the writer can not but believe that the water supply estimated by Mr. Himes is not entirely adequate for the supply of such a canal as has been proposed. If such a canal is constructed, the experience gained in the last seventy-five years ought to teach the danger of small economies in designing the water supply. Experience shows that canal water supplies must be made ample, as otherwise a shortage will result sooner or later.

In a paper on an enlarged waterway between the Great Lakes and the Atlantic seaboard, published by William Pierson Judson, the water supply of the summit level of the Oswego-Mohawk-Hudson route is discussed at length. Mr. Judson considers that it would be entirely proper to take whatever deficiency there might be from the

[1] See report on the Enlarged canal via the Oswego route, by Albert J. Himes, resident engineer, Eastern division, New York State canals. In Ann. Rept. State Eng. and Surv. for the fiscal year ending September 30, 1895.

head waters of the Black River, reservoirs in addition to those now existing being constructed on the Beaver and Moose rivers, tributary to the Black, for the purpose of furnishing this water. He recognizes that the item of adequate water supply for such a canal is vital, and frankly states that if surveys and thorough investigations were to show that the demand for water for such a canal is beyond the capacity of the sources of supply, then the Oswego-Mohawk-Hudson route would be shown to be impracticable, although, as an alternative proposition, he states that it would be entirely practicable to supply the summit level of such a canal from Lake Erie. This, it is pointed out, can be accomplished by a feeder branch taken from the present Erie Canal near Macedon, 12 miles west of Newark, where Erie Canal is now 35 feet above the Rome level. The proposed feeder, instead of stepping down, as does the Erie Canal, can be swung off to the south on higher ground at the necessary elevation, passing along the south side of Clyde River and crossing Seneca River near the Cayuga Lake Outlet. Seneca River is narrowest here, and the feeder could be carried across it in an open trunk on a 40 to 50 foot trestle about 2 miles long. Such a feeder 8 feet in depth and 38 feet in bottom width would carry 1,000 cubic feet per second, or enough, according to Mr. Judson, to meet the assumed needs. On this point, however, the author differs; he can not but think that the summit-level supply as estimated by Mr. Judson at 1,000 cubic feet per second is much too small. Mr. Judson's discussion of the deep-waterway question is further open to criticism in that he does not adequately recognize the economic value of the New York water supplies for use in manufactures.

In a paper read before the American Society of Civil Engineers in 1884, Elnathan Sweet proposed a ship canal through to Lake Erie via present line of the Erie Canal, except that its alignment be rectified to follow the line of the feeder, as proposed by Mr. Judson. The difficulties of making a ship canal on this line are, however, so great that many engineers have considered it absolutely out of the question.[1]

A canal on the Oswego-Mohawk-Hudson route 28 to 30 feet in depth, with corresponding surface and bottom dimensions, will probably absorb all available water of central New York, as well as a considerable portion of Black River. The water powers on Mohawk River at Cohoes will necessarily be made subservient to the exigencies of such a canal, although Mr. Judson, in the paper already referred to, has pointed out how valuable these water powers would be for seven or eight months of the year to the manufacturing cities of the Mohawk Valley. Under this head we may, however, inquire as to how the water power for only seven months of the year would be of any special

[1] See Radical enlargement of the artificial waterway between the Lakes and Hudson River, by Elnathan Sweet: Trans. Am. Soc. Civil Eng., Vol. XIV, pp. 37-139.

value to the city of Cohoes, where, owing to the kind of manufacturing, continuous power three hundred and ten days in the year is required. Cohoes had a population in 1890 of 22,509, and is stated to have grown considerably since, so that in 1897 the population is probably in excess of 25,000. This great development is a result of wise management of the water power, without which there is no reason to suppose that the area on which the city stands would have any greater value than that of the surrounding farming region. A proposition to interfere seriously with the water power at Cohoes can only be looked on by the author as most extraordinary. Indeed, not the least extraordinary feature of the present agitation for ship canals across the State of New York is the entire lack of appreciation—so far as the discussion indicates—of the value to the State of New York of its inland waters.

Aside from the report of Major Symons, the discussion has thus far apparently proceeded on the supposition that the taking of inland waters for navigation purposes was a matter on a par with the taking of agricultural lands for right of way, the economic value of the water for power purposes and the resulting effect on the internal development of the State having thus far been almost entirely ignored.

What the people of the State of New York need to consider first of all is whether the inland waters are not now worth more for manufacturing than they can possibly be worth for navigation purposes. If after investigation it is shown that the water will produce greater income to the people of the State in manufacturing than it will in operating such a canal, then from mere commercial considerations the people ought not to consent to the construction of such a canal. Without having the data at hand for a full discussion, the author, after giving the matter careful consideration, is of opinion that the State of New York can not afford to forego the possibility of developing its manufacturing interest in order to furnish water for the summit level of the proposed Oswego-Mohawk-Hudson deep-water canal. At any rate we should know just what results may be expected before embarking in the enterprise. If, however, after full investigation it appears that the canal water supply can be obtained and the manufacturing interests protected, no reasonable objection can be urged.

In order to justify the construction of the ship canal as a commercial proposition, the saving on the transportation of an estimated annual tonnage of 24,000,000 tons over the cost of its transportation by existing means and methods must, at least, equal the interest on the cost of the canal plus the annual cost of maintenance and operation. The first cost is taken at $200,000,000, with the maintenance at $2,000,000 per year. Assuming an interest charge of 3 per cent, the annual interest plus the maintenance becomes $8,000,000, which sum

ɔresents the annual expense of the proposed ship canal connecting ıe Great Lakes with the Atlantic seaboard. As regards the State of ew York, there should be added to this amount a sum representing ıe decrease in wealth in central New York due to the absorption of ıe inland waters of the State away from manufacturing interests in ,vor of navigation interests. As a rough estimate the author places ıch decrease at not less than $5,000,000 per year, although the ɔcrease would probably be much greater than this, but in the absence data for full discussion he places it at a conservative figure, which n not well be gainsaid. On the other hand, if the international St. ıwrence-Champlain-Hudson route were to be constructed, not only ɔuld this source of loss be entirely eliminated, but since that plan oposes to deliver water from St. Lawrence River into Lake Chamıin, and thence by a through cut from Lake Champlain to Hudson ver, there would be delivered into Hudson River a considerable antity of water which would be available for power at Saratoga m, Mechanicville, and Troy. This ship-canal project thus increases ther than decreases the productive capacity of the State. Moreover, is probable that the St. Lawrence-Champlain-Hudson route can be ıstructed somewhat more cheaply than the Oswego-Mohawk-Hudson ıte.

Without wishing to present the foregoing as in any degree a final ıclusion, the author can not but think that it is the broad view to ɜe of the question. At any rate, this view presents a line of invesıation which ought to be pursued to a conclusion before the final cision is made.

LOSS OF WATER FROM ARTIFICAL CHANNELS.

The large amount of canal construction in New York State has ɜessitated, in order to provide ample water supplies, the collection considerable information as to the various sources of loss of water to ıich artificial channels are subject. Some of the more interesting ;ults may be briefly referred to.

The original Erie Canal was constructed with the water surface 40 ɜt wide, the bottom width 28 feet, and the depth 4 feet. In 1824 ɜasurements of the loss from filtration and evaporation were made Mr. John B. Jervis on the eastern division and by Mr. David S. ıtes on the western division. Mr. Jervis states that his measure-ɜnts were made in the original Erie Canal, between the first locks low the village of Amsterdam and the aqueduct below Schenectady, listance of 18 miles. This section was constructed mainly through alluvial soil, containing a large portion of vegetable matter. In me places this soil was very leaky, owing probably to the decay of ots, although the greater portion retained water very well. There ıs a considerable quantity of gravel and slaty soils. He states that

the quantity of water lost in this 18-mile section was very uniform, and averaged 125 cubic feet per mile per minute.[1]

Mr. Bates states that his measurements in 1824 showed that a mile of new canal, such as Erie Canal then was between Brockport and Ninemile Creek, would require 1.7 cubic feet of water per mile per second in order to supply the losses from filtration, leakage, and evaporation.[2] The following are some of the details of Mr. Bates's measurements in 1824:

On 79 miles of canal and feeder, comprising 20 miles of canal from Rochester to Brockport, 57 miles from Rochester to Cayuga, and 2 miles of feeder, the supply was 8,000 cubic feet per minute, or 121.66 cubic feet per mile per minute. The months are not stated, although it may be inferred that these observations are averages of the navigation season.

Mr. Bates further states that in August, 1824, he found a total use for the 20 miles from Rochester to Brockport of 2,100 cubic feet per minute, equal to 105 cubic feet per mile per minute. This section of the original Erie Canal was considered to be entirely free from leakage at the structures, and the measured losses are therefore taken as those due only to percolation, absorption, and evaporation.[3]

In August, 1839, Henry Tracy and S. Talcott, acting under instructions from W. H. Talcott, resident engineer of the fourth division of Genesee Valley Canal, made a series of observations along the line of Chenango Canal, with a view of determining the evaporation, filtration, and leakage at the mechanical structures, and whatever else might be useful in the designing of the water supply of the summit level of Genesee Valley Canal.

For the purposes of the measurements they selected a portion of the canal extending from the north end of the summit level to Erie Canal, 22 miles in length, on which the total supply on August 31 was found to be 39 cubic feet per second. The leakage and waste at aqueducts, waste weirs, and at lock No. 1 at the northern end, were found to be 15 cubic feet per second, thus leaving the evaporation and filtration on 22 miles at 24 cubic feet per second, equivalent to 1.09 cubic feet per mile per second. It may be observed, however, that a measurement made at the end of August would probably not show a maximum of either evaporation or absorption by vegetation. Estimating these elements at the maximum, we may assume from 1.33 to 1.67 cubic feet per mile per second as a more reliable quantity than the 1.09 cubic feet per mile per second here actually observed.

[1] Report of John B. Jervis to the canal commissioners, on the Chenango Canal. Ann. Rept. Canal Com. (1840). Ass. Doc. No. 55, p. 54.

[2] Report of David S. Bates to the canal commissioners, on the Chenango Canal (1824). Ass. Doc. No. 47, p. 31.

[3] See report of F. C. Mills in relation to the Genesee Valley Canal (1840). Ass. Doc. No. 26, p. 8. See also report of W. H. Talcott in the same document. These two reports contain a summary of all that had been done in the way of measurements of the various losses now under discussion up to that time, as well as a number of references to foreign data.

Messrs. Tracy and Talcott also measured the leakage and waste at the various mechanical structures, etc., which were as follows: Leakage at structures, 220 cubic feet per minute; waste at waste weirs, 204 cubic feet per minute; leakage at lock No. 1, at the north end of the section, 479 cubic feet per minute. This amount, Mr. Talcott remarks, was so much greater than at any other lock on the canal as to induce the belief that the gates were not properly closed at the time of measurement. At lock No. 69 on the same canal, the leakage was 382 cubic feet per minute from an 8-foot lift.

Mr. Talcott's report is very able, and presents forcibly all the data at hand at that time. It may be said that the data which he gave fixed the following quantities as fairly covering the various losses to which artificial waterways of the dimensions of the original canals of this State are subject:[1]

(1) Loss by filtration, absorption, and evaporation, 100.0 cubic feet per mile per minute. With retentive soils this could be reduced to from 60 to 70 cubic feet per mile per minute. Mr. Talcott fixed on 66 cubic feet per mile per minute for Genesee Valley Canal, which was largely built through heavy soils; but this was subsequently found too small.

(2) Leakage at mechanical structures; for locks of 11 feet lift, 500 cubic feet per minute; for leakage and waste at each waste weir, 30 cubic feet per minute; for a wooden-trunk aqueduct, an amount depending on the length of the structure, but as an average, 0.35 of a cubic foot for each linear foot of trunk may be taken.

In response to a resolution of the canal commissioners of April 12, 1841, Mr. O. W. Childs, then chief engineer of Erie Canal, prepared a report on the water supply of the western division with reference to the enlargement then in progress.[2] In this paper Mr. Childs gives the results of measurements made by himself in 1841 of losses from filtration, absorption, evaporation, and leakage on the original Erie Canal between Wayneport, in Wayne County, and Pit Lock, which corresponded to lock 53, near Clyde, of the present canal. He also gave the result of measurements made by Alfred Barrett between Pittsford and Lockport.

Mr. Childs's measurements were for a section of the canal 36.02 miles in length. On the Palmyra level, for a distance of 8.34 miles, where the soil is open and porous the measurements showed a loss of 1.81 cubic feet per mile per second. On the Clyde level with a more retentive soil the losses from filtration, absorption, and evaporation were, for a distance of 27.68 miles, only 0.59 cubic feet per mile per second. The entire loss, including leakage, was, for the whole distance, 1.40 cubic feet per mile per second. These measurements were made for

[1] The quantities here given apply to canals 40 feet by 28 and 4 feet deep, and with locks 90 feet in length and 15 feet in width and 8 to 10 feet lift.

[2] See Supply of water required for the canal between Lockport and the Seneca River, by O. W. Childs: Ann. Rept. Canal Com. (1848). Ass. Doc. No. 16, pp. 141-175.

a term of thirty-three days, from July 30 to August 31, inclusive. Measurements were also made in June, early in July, and in the following October, from which the conclusion was derived that demands were greater and the supply less for the time during which the foregoing observations were taken than during any other portion of the season.

Mr. Barrett's measurements were made at various points on the original canal between Pittsford and Lockport, and repeated each day from July 17 to September 30, inclusive. They showed an average loss for the whole period of 73.0 cubic feet per mile per minute. Assuming the same ratio of loss between Pittsford and Wayneport, there resulted, for the entire distance of 122 miles from Lockport to Pit Lock, an average loss of 67 cubic feet per mile per minute. Mr. Childs states that an addition to the foregoing quantity should be made as an allowance for springs and several small streams entering the canal which could not be measured. Making such additions he concludes that 1.42 cubic feet per mile per second should be taken as the total quantity consumed on the 122 miles of canal under consideration, which is equivalent to a total of 173 cubic feet per second. It is stated in the original reports that the supply of water was ample for all the purposes of navigation during these measurements.

Comparing Mr. Childs's measurements of 1841 with those made by Messrs. Jervis and Bates in 1824, one point of great practical utility is strongly brought out, namely, as to the excess of loss of water in new canals over those some time in use; thus Mr. Bates found in 1824, on the same reach of canal as was measured by Mr. Childs in 1841, a total loss of from 1.68 to 1.75 cubic feet per mile per second. It may be assumed that the springs and streams allowed for by Mr. Childs were delivering into the canal in 1824 the same as in 1841, at least 0.17 to 0.25 cubic feet per mile per second. We have, then, as the total supply in 1824 from 1.92 to 2.00 cubic feet per mile per second. Adopting the latter figure as a maximum to compare with Mr. Childs's figure of 1.42 cubic feet per mile per second, as found in 1841, the conclusion is reached that the decrease in the loss by filtration—due presumably to the gradual silting up of the bottom—is something like 0.75 cubic feet per mile per second.

This conclusion may be applied to the conditions of the Erie Canal enlargement now in progress, in which it is proposed to excavate 1 foot from the bottom of many of the levels. The effect of this will be to remove the silt accumulations of many years, thus placing the bottom of the canal, as regards porousness and consequent percolation and filtration loss, in the same condition as when constructed. This consideration alone indicates the necessity of making the water supply of the enlarged canal liberal in order to answer the demands of the first few years while the bottom is again attaining a fixed condition.

Under this head it may be remarked that the experience of seventy-five years in the operation of the New York State canals has thoroughly shown the futility of any attempt at excessive economy in water supply. In the absence of systematic information as to yield of streams, the general tendency has been to overrate the summer flow, with the result of shortage frequently at points where the supply was believed to be ample. The chief sources of such shortage may be enumerated as follows:

(1) The great variation in the yield of drainage areas from year to year, by reason of differences in the rainfall, humidity, and temperature.

(2) The cutting off of forests, which has increased somewhat the spring-flood flows and decreased the summer flow.

(3) The systematic drainage of large areas, which has also tended to increase the flood flows and decrease the summer flows.

(4) The growth of aquatic plants on long levels and the formation of sand bars in the canal, which have tended to decrease the amount passing.

Among minor sources of loss, evaporation and absorption by growing plants may be mentioned, both of which vary somewhat in different years, although neither can be considered a serious source of loss. A number of other measurements of the losses from the original Erie Canal are recorded in the reports, but the foregoing are sufficient for present purposes.[1]

A study of all the measurements in detail shows that in an artificial channel of the dimensions of the original Erie Canal, and constructed on the American system, there should be provided at least 80 to 100 cubic feet per mile per minute, exclusive of water for filling and for lockages.

Using the data of the measurements of 1841, Mr. Childs arrived at the water supply of the enlarged canal of that day in the following manner: It was assumed that the loss by filtration through the bottom and sides of the canal would be as the square root of the pressure or depth of the water, and as the area of the surface pressed. Proceeding on this assumption, he computed the quantity required to supply the losses from filtration, leakage, and evaporation (in the enlarged canal, 1840 to 1860), at 3.17 cubic feet per mile per second. This figure was subsequently substantially adopted for the entire enlarged canal, and, with the exception of a few special cases, is still in use.

Adding the amount required for lockages at lock 53, Mr. Childs placed the entire supply for the western division, from Lockport to the east end, at 3.48 cubic feet per mile per second, or at a total of 424 cubic feet per second for 122 miles of canal.

The canal enlargement now in progress contemplates an increase in

[1] Ass. Doc. (1840), No. 96, p. 26; also Ass. Doc. (1842), No. 24, p. 37.

depth from 7 to 9 feet. Taking into account the results of the measurements on the original Erie Canal, as well as those made by Mr. Childs on the enlarged canal of 1840 to 1860, it has been concluded that the proper figure for water supply on the western division, to which the studies thus far specially refer, should be taken at from 4.17 to 4.5 cubic feet per mile per second.[1]

As to the use of water for lockage, leakage of gates, drawing and swelling of boats, and for turbine water wheels to operate gate-lifting machinery, reference may be made to the Annual Report of the State Engineer and Surveyor for 1889, where data covering these different points may be found.

Ordinarily, the flow of a natural drainage channel increases farther down the stream. The outlet of Skaneateles Lake, however, appears to present an opposite example—that is to say, there is less water flowing at the mouth of the stream than at its head. This statement is derived from a report on the water supply of the middle division of Erie Canal, made in 1862,[2] according to which measurements were made by Mr. S. H. Sweet of the flow through the natural channel of the Skaneateles Outlet, 10 miles in length, discharging into the Erie Canal at Jordan. Mr. Sweet's measurements, which were continued through the dry season of 1859 and the entire season of navigation, apparently indicated a loss of water of more than 3,000 cubic feet per minute in this natural channel. The detail, however, is lacking, and probably before accepting this as a fact the measurements made in 1859 should be verified.

USE AND VALUE OF WATER POWER.

WATER POWER OF ERIE CANAL.

When Erie Canal was first constructed the policy was adopted of leasing the so-called surplus waters for power purposes. Under the terms of the act of 1825, leases were made during 1826 and subsequent years to a number of persons at Black Rock, Lockport, and other localities.

POWER AT BLACK ROCK.

The granting of these leases and the resultant development of large manufacturing interests at several points have raised certain economic questions which will now be briefly discussed. The water power at Black Rock, for which several leases were granted, may be first mentioned. This power is created by the difference in level between the water in Erie Canal and Black Rock Harbor and that in Niagara River outside the harbor wall, this difference of water level

[1] The foregoing statements in regard to measurements of water supply of Erie Canal are abstracted from a report on the water supply of the western division of the Erie Canal by the author, and are to be found in Appendix I to the Ann. Rept. of the State Eng. and Surv. for the fiscal year ending September 30, 1896.

[2] See Ann. Rept. of State Eng. and Surv. for the fiscal year ending September 30, 1862, pp. 42-46.

amounting to from 4 feet to 4.5 feet. As measured in the spring of 1896, at a point near the ship lock, it was about 4 feet. According to the report of the assembly committee of 1870, as referred to in the footnote, there were formerly ten mills in operation at Black Rock, using 2,744 second-feet of water. The power developed by these mills, and all operating at full capacity, is estimated at not exceeding 520 horsepower. Owing to the decline of the milling business in New York State a number of these mills have passed out of existence.

The four mills still in existence require about 1,200 cubic feet of water per second to operate them at the full capacity of the wheels now in place.

The use of water by the Black Rock mills has always been a detriment to navigation. When all were running the amount of water actually drawn through the canal and harbor for their supply, and for the supply of the canal to the east of Buffalo, was fully 3,300 cubic feet per second.[1]

When all the Black Rock mills were in operation the great draft of water so obstructed the navigation that the legislature finally authorized the construction of a division wall in Black River Harbor, by which it was expected that the water supply for the mills would be entirely taken from the harbor, leaving the channel of the canal pretty nearly free for the purposes of navigation; but after the greater part of the wall was completed it was ascertained that because of the silting of the upper harbor with sewage mud, as well as drifting sand from the lake, there would be difficulty in obtaining the full supply for the mills through the harbor, without extensive dredging. The division wall was, therefore, never completed, two gaps, amounting, in the aggregate, to several hundred feet, having been left below Terry street. There was thus an expenditure of about $350,000 for the benefit of the milling interests which is entirely without effect for lack of completion. Under the present conditions, however, of entire decline of the Black Rock milling interests, there is, of course, no reason why the wall should be completed, and the matter is discussed here at all merely for the purpose of bringing out clearly the struggle between the navigation interests and the manufacturing interests, which has been in progress in New York State for the last seventy-five years.

POWER AT LOCKPORT.

At Lockport the construction of the Erie Canal through the mountain ridge created a fall of 58 feet at a single point, and since the use of water for lockage purposes is only a small part of the whole flow. The balance required to feed the canal to the east of Lockport is necessarily discharged around the locks into the lower canal by means of

[1] The assembly committee of 1870 give the following figures as then applicable: Lower Black Rock mills, 1,887 second-feet; upper Black Rock mills, 858 second-feet; for supply of canal, 583 second-feet; total, 3,328 second-feet.

sluiceways. Under the provisions of the laws of 1825, a public auction was held in the village of Lockport, in the fall of that year, and the right to use this surplus water sold to Messrs. Richard Kennedy and James H. Hatch, whose successors at the present day constitute the Lockport Hydraulic Power Company.

Lockport has usually been considered more purely a result of the canal development than any other point in western New York, for the reason that while nearly all other towns in the region had some growth before the Erie Canal was located, it was only in 1821, after the present location for the canal had been definitely decided on, that the nucleus of a village was formed here by the contractors and their workmen employed on the canal. In 1820 there was no frame house or barn within 5 miles of Lockport, and there were less than 600 acres of cleared land in the 4 square miles, of which the city of Lockport is now the center. Moreover, there are no natural advantages which would have naturally led to the growth of an important town at this point. The water supply of the region is so deficient that even to this day the city takes its public supply from Erie Canal, which is grossly polluted with sewage from the city of Buffalo. It may be considered, therefore, that the city of Lockport owes its existence entirely to the creation by Erie Canal of a large water power at this point.

When once started, however, under the impulse of the canal development, Lockport grew rapidly until, in 1829, with a population of 3,000, it was incorporated as a village, and in 1865 as a city. The population in 1890 was 16,038; in 1897 it is estimated at over 17,000.

The total investment in manufacturing plants at Lockport dependent on the Erie Canal water supply amounts to $2,531,000. The total number of establishments is 33, employing 1,880 operatives. The total power now in use on Erie Canal proper is 2,625 net horsepower.

A short distance to the east of the foot of the locks a small stream known as the West Branch of Eighteenmile Creek crosses under the canal. This stream, although having a drainage area of only 1 or 2 square miles to the south of the canal, has cut a deep valley with rapid fall for a considerable distance to the north of the canal. In order to provide for discharging the surplus waters from the canal, an overflow into Eighteenmile Creek was constructed at an early day. A mill was also permitted to take water from the lower level and discharge its tail-water into the creek. Finally the Jackson Lumber Company was permitted to construct a sluiceway on the towpath side, through which it drew for many years about 600 second-feet, and which was all discharged into Eighteenmile Creek. Complaints having frequently been made that boats were drawn against this sluice on the towpath side, the superintendent of public works, in 1892, granted a formal permit to the Jackson Lumber Company to construct a sluice and subway under the canal bottom, by which this water is now drawn from the berme side. Under this permit a substantial

masonry sluice was constructed in 1893. In the meantime the Jackson Lumber Company has gone out of existence, and this water power has passed into the hands of the Traders' Paper Company, which now occupies the site with its pulp mill No. 1.

West Branch of Eighteenmile Creek descends about 175 feet within the limits of the city of Lockport, of which 148 feet have been utilized for power during recent years.

The following are the companies now using power on this creek and the horsepower used by each:

Power utilized on West Branch of Eighteenmile Creek.

	Horsepower.
Traders' Paper Company	1,060
Lockport Paper Company	230
Niagara Paper Company	115
Westerman and Company	320
Cascade Pulp Company	925
Cowles Smelting Company	1,185
Total	3,835

The output of the establishments on West Branch of Eighteenmile Creek is about $2,000,000 a year; but this sum includes the output of the Indurated Fibre Company, which, while operating by steam power, depends largely for a supply of pulp on the Cascade Pulp Company. In any case, the figures show the magnitude of the manufacturing interests which have been fostered in the valley of West Branch of Eighteenmile Creek, by discharging into that stream about 300 second-feet of water from the Erie Canal.

With 2,625 net horsepower in use on the canal proper, and 3,835 on West Branch of Eighteenmile Creek, the total actually in use at Lockport, and dependent on Erie Canal for its water supply, is 6,460 net horsepower.

No statements as to the value of the annual product of the manufacturing establishments on the raceways of the Lockport Hydraulic Power Company have been given. It is therefore impossible to state accurately the value of the total annual product at Lockport. As several of the establishments there are very extensive, including the Holly Manufacturing Company, it may be assumed that the annual output of this portion of the Lockport manufactories has a value, at least, of $1,000,000; hence we reach a total value of the annual product for the whole city of about $3,000,000.

The annual rental paid to the State, under the terms of the original lease, is only $200. At first sight it appears that there is here a most marked case of what could only be termed blundering on the part of State officials, although on analyzing the matter it is found that this extreme view is hardly correct. In the first place it must be remem-

bered that this lease was granted not only by authority of an act of legislature, but was only granted after a public auction had been held, at which Messrs. Kennedy and Hatch were the highest bidders. As already shown, had not the special conditions created by Erie Canal existed at Lockport, there would, in all probability, have been no thriving city at that point, but the area on which Lockport now stands would have been farming land, with no more value than now attaches to farming lands in the adjoining township of Lockport.

In order to show the results of this lease, at $200 a year, a study has been made of the growth of Lockport from the year 1865, when Lockport became a city, to 1896. It appears, for instance, that the valuation of the city has increased from less than $3,000,000 to over $6,700,000, and that the total State tax collected up to and including the year 1896 has amounted to over half a million dollars. If this had remained as a small farming community the State tax would probably not have been more than 3 per cent of this amount. Using this tax return as a basis, it has been computed that there has been an actual increase of wealth to the people of the State by the existence of Lockport of over one and a half million dollars, not including in this the actual increased value of the city itself. The conclusion is drawn that the benefit to the State at large has been very great on account of this expenditure for internal improvement, irrespective of questions of navigation.

POWER AT MEDINA.

The Oak Orchard feeder and the water power at Medina present somewhat different points for consideration from those at Lockport.

About 1820 the canal commissioners caused a cut-off channel to be constructed through Tonawanda swamp between Tonawanda and Oak Orchard creeks, whereby the early summer flow of Tonawanda Creek is diverted into Oak Orchard Creek. Oak Orchard Creek passes under the Erie Canal at Medina, and the original feeder channel at that place was an artificial channel leading from a dam thrown across the creek and entering the canal near West Branch of Oak Orchard Creek at Medina. At some period subsequent to 1823 a race way was constructed by private parties leading from a second dam higher than the feeder dam and conducting water into the central part of the village, where, after it is used, it is finally allowed to pass into the canal. During the enlargement of 1836 to 1862 the water-surface level of the canal at Medina was raised, and inasmuch as this change necessitated raising the feeder dam somewhat, it was finally concluded to discontinue the feeder and depend entirely on the race way for such supply as the canal might receive at this point.

Oak Orchard feeder has been considered as furnishing about 27 cubic feet of water per second to the canal, although measurements made in 1850 show about 37 cubic feet per second. Since then the clearing up of forests and the drainage of Oak Orchard and Tona-

wanda swamps have tended to reduce materially the low-water flow until it is probably less than 27 cubic feet per second. Moreover, for the future, the dry-weather yield from this drainage area may be expected to be somewhat less than in the past, because of the deepening of the channel of Oak Orchard Creek and of the crosscut authorized by the laws of 1893. The act provided for deepening the channel of Oak Orchard Creek from a point 2½ miles below where Tonawanda Creek enters the Oak Orchard and for the cleaning, improving, widening, and deepening of the channel of East Branch of Oak Orchard Creek. This work has been done as a sanitary measure, and its effect will probably be to run the water out of the swamps more rapidly in the spring, thus materially decreasing the dry-weather flow.[1]

According to a statement furnished by Mr. A. L. Sweet, president of the Business Men's Association of Medina, the number of operatives employed in 1896 in manufacturing enterprises dependent on water power at Medina was 515; the amount of capital invested in establishments actually in operation was $371,000, while the value of the annual product of the same establishments was $575,000. These figures do not include the Medina Falls flouring mill, which was idle at the time these statements were made.

The total developed water power at Medina, on the race way and on the Oak Orchard Creek, is estimated at 827 horsepower, which includes the wheels at Medina Falls flouring mill. Deducting these wheels, amounting to 338 horsepower, the total actually in use in 1896 is 489 horsepower. The use of water at the establishments on the creek varies from 110 cubic feet per second to 49 cubic feet per second, the former quantity being due to the Medina Falls flouring mill, where the head is 33 feet. Relative to the fine power at Medina Falls, it may be stated that it is improbable, considering the amount of power available at this location, that it will remain unutilized for any great length of time. The trouble at Medina Falls flouring mill is the same as that affecting the large flour mills at Black Rock and other places in New York—the competition of cheap grain and transportation from Western mills.

Without going into the historical part of the subject, it may be said that the mill owners at Medina claim that by reason of the granting of a right of way for the cut-off between Tonawanda and Oak Orchard creeks, and the gift of 100,000 acres of land to the canal fund by their original grantor, the Holland Land Company—a part of the consideration for which was an improvement of the water power of Oak Orchard Creek—they have an equitable right to the use of the water of the feeder. If, therefore, the effect of the drainage authorized by the laws of 1893 has been to decrease the low-water flow of

[1] For extended account of Oak Orchard Creek and its relations to the feeder, see Report on the drainage of the Oak Orchard and vicinity streams, in the Fourth Ann. Rept. of the State Board of Health (1883), pp. 43-116.

Oak Orchard Creek, it is maintained that the mill owners are entitled to enough water from the canal to make good the deficiency.

There are a number of other points on the Erie Canal where water powers have been fostered under the provisions of the laws of 1825, but lack of space precludes discussion of that phase of the subject.

SELLING PRICE OF WATER POWER.

The principal places in New York State at which hydraulic developments have thus far been made for the purpose of selling power are Oswego, Cohoes, Lockport, and Niagara Falls. At Oswego the power on the east side of the river is owned by the Oswego Canal Company, the development being by a canal 4,000 feet long, with an average surface width of 60 feet and a depth of 6 feet. The water from this canal is dropped into Oswego River at the level of Lake Ontario. The working head is from 18 to 20 feet, although with high water in the canal and low water in Lake Ontario, the working head becomes somewhat greater.

The State controls the first right to the flow of Oswego River in order to maintain slack-water navigation in the pool above the dam at the head of the Oswego Canal Company's race way, all water not needed for canal purposes being equally divided between the Oswego Canal Company's race on the east side and the Varick Canal on the west side. The Oswego Canal Company gives a 999-year lease of water, but without land for location of buildings. A water right on this canal is called a run, meaning, probably, the amount of water required to drive a run of stone, a run of water being taken at 11.75 second-feet which, under the ordinary working head of 20 feet, will, at 75 per cent efficiency, produce 20 horsepower. There are assumed to be 32 first-class runs, the rental for which is $350 a year for each run. At this price the cost of a horsepower a year, with 75 per cent efficiency, becomes $17.48, or the cost of a gross horsepower a year becomes $13.11. There are also 32 second-class runs, of which the rental varies from $250 to $300 a year for each run. Further, there are surplus runs which are rented at a little over one-half of the rental charged for first-class runs. In case of a shortage of water the surplus runs are shut down successively, beginning with the most recent leases; after this the second-class runs share equally with one another in reduction; and finally, in case of extreme shortage, the first-class runs are similarly cut down.

The Varick Canal on the west side of the river controls one-half of all the water not needed for navigation purposes, the same as the Oswego Canal Company's canal on the east side. In order that the water may be divided equally between these two canals both have the same aggregate waterway at the head gates, and by gages on both sides, which are examined whenever necessary, it can be seen whether one canal is drawn below the other, and the gates changed accord-

ingly. On this canal there are recognized 50 first-class runs, 17 second-class, and an unlimited number of third-class. For first-class runs the rental is from $250 to $300 per annum; for second and third class it ranges from $125 to $150. By a decree of the supreme court, dated August 21, 1875, a run of water on the Varick Canal ranges between 28 cubic feet per second, under a head of 12 feet, and 25 cubic feet per second, under a head of 13 feet. The actual working head is, however, ordinarily only about 10 feet, so that on the foregoing basis a run of water may be taken as 33.3 cubic feet per second. At the price of first-class runs of from $250 to 300, and with 75 per cent efficiency, the cost per horsepower per annum varies from $8.80 to $10.56, a run on the Varick Canal being equal to 33.3 cubic feet per second on 10 feet head, an amount of water which yields 37.9 horsepower under that head.

As to the difference in cost of water on these two canals at Oswego, it may be pointed out that the Oswego Canal Company's race has a substantial advantage over the Varick race, in that it extends to the harbor, enabling vessels to come directly alongside of the mills. Moreover, the division of water rights is such that a first-class run of water can always be depended on along the Oswego Canal Company's race, but can not on Varick Canal.[1]

At Cohoes we have the great power development built up by The Cohoes Company, which has, by careful management of the water power, built up at this place a fine manufacturing city of 25,000 inhabitants. Lack of space will not permit description of this development in detail.

The Cohoes Company not only owns all of the hydraulic canals, but also the land adjoining the canals. It gives to manufacturers a perpetual lease of land and water, the entire property leased remaining subject to a rental of $200 per year per mill power. On this basis the land is regarded as donated and the rental applies only to the water power. Formerly, the standard for measuring water was 100 square inches, to be measured through an aperture in a thin plate 50 inches wide, 2 inches deep, and under a head of 3 feet from the surface of the water to the center of the aperture; but in 1859 a series of measurements were carefully made under the direction of the late James B. Francis, using an old canal lock as a measuring chamber. These measurements showed that the old standard corresponded to about 5.9 cubic feet of water per second. As a result 6.0 cubic feet of water per second, under 20 feet head, was taken as a new standard constituting a mill power. On this basis a mill power is equivalent to 13.63 gross horsepower, which, at $200 per mill power per annum, costs $14.67 per gross horsepower per annum. At 75 per cent efficiency the annual rental for water for net hrrsepower becomes $19.57. In

[1] For additional detail of the water power at Oswego, see Report of Water Power of the United States, Tenth Census, Vol. I, pp. 24-27.

regard to just what is paid for by the annual rental, both at Oswego and Cohoes, it may be remarked that the foregoing prices are for water in the race way, the company maintaining the dams, head works, and main race ways, the lessee taking the water at the face of the race way and maintaining his own head gates, flumes, bulkheads, wheels, and any other appliances necessary for utilizing the water in the production of power.

The water power at Lockport, owned by the Lockport Hydraulic Company, is formed by the drop of the surplus water of the Erie Canal through a distance of 58 feet. A run of water at Lockport does not appear to be very well defined, but the rental charge ranges from $12.50 to $16.67 per effective horsepower. So far as known to the author, just what constitutes an effective horsepower has not been defined.

At Niagara Falls the rental price of undeveloped hydraulic power has been fixed at from $8 to $10 per gross horsepower per annum, the party renting the power taking the water at the face of the head race and making its own connection with the discharge tunnel. Electric power by a two-phase alternating current as it comes from the generator is sold in blocks of 2,000 or 3,000 horsepower, at $20 per net horsepower per annum, the purchasers furnishing transformers, motors, and all other electric appliances. In small blocks the price has been fixed somewhat higher.

A small amount of power has also been sold at different times at Rochester, but since the power at this place is nearly all held by manufacturers who use it at first hand, nothing like a fixed price has been made at Rochester. Generally, power rented has been in small quantities and in connection with floor space, the rental price being really for floor space with small power furnished. Reckoning on this basis, small powers have frequently been rented at Rochester at as high a price as $100 per horsepower per year, this being for power on the shaft, all expenses of maintaining wheels, transmission shafts, etc., being borne by the owner.

The electric companies at Rochester furnish electric power in small blocks at 3 cents per electric horsepower per hour, which, on the basis of ten hours a day and three hundred and ten days a year, becomes $93 per electric horsepower per annum.

STATE OWNERSHIP OF INLAND WATERS.

Without going into a detailed discussion as to State ownership of inland waters in New York, attention may be called to a few of the more essential facts. The absolute ownership by the State of the beds and banks and water flowing in the Hudson and Mohawk rivers has been confirmed by repeated decisions of the highest courts of the State. By reason of the peculiar circumstances of the early settlement of the country the English common-law rule applies to all the

streams of the State except the Hudson and Mohawk, from which it results that all other inland streams are owned to the thread of the stream by the abutting proprietors.

Owing to a confusion of ideas—largely on the part of citizens appointed to the position of canal appraisers, where they exercised to some extent judicial functions—there arose in the early days a broad claim of State ownership to inland waters tributary to the Erie canal. The claim was, however, finally decided in the negative. This condition has, in the case of Genesee River, worked great injustice to the riparian owners, in that under a claim of temporary diversion and an assumed right of permanent appropriation by a statutory enactment declaring the stream a public highway no adequate damages have ever been paid for the diversion actually made from the stream for many years.

A similar lack of appreciation of the real relations of the State to individual citizens in regard to common rights in running streams has apparently led to the enunciation of a course of action on the part of canal officials which, to some extent, in effect confiscates private property.

In the case of Black River the State has extensively adopted the method of compensation in kind rather than compensation in money. In view of the large development of storage on Black River, under the provisions of chapter 181 of the laws of 1851, in excess of what is really required there actually to make good the diversion during the low-water season, and, further, by reason of the State's finally placing the management of the principal reservoirs there in the hands of a commission of owners and users of water power, as provided by chapter 168 of the laws of 1894, it may be assumed that as regards Black River, the State has adopted the policy of conservation of water power, such policy being in line with the best thought of the present day.

In the case of Skaneateles Lake there was for several years a struggle between the State and the city of Syracuse as to the right of that city to obtain the municipal water supply from Skaneateles Lake, the extraordinary feature of that controversy being the assumption on the part of the agents of the State that the municipalities of the State, even in the important matter of public water supply, had no rights which the State apparently was bound to respect.

It may be pointed out that the considerable diversity of law and policy, in regard to the rights of riparian owners thus shown to exist in the State of New York, can not operate other than to discourage the full development of the State's natural resources. What is greatly needed, therefore, in the State of New York is a consistent policy of some sort as between the State, the navigation, the manufacturing, and all other interests, in which each shall receive proper consideration. Thus far navigation interests have usually been placed first,

although it is clear that had there been a more thorough balance, the State as a whole would have been considerably in advance of its present position.

FUTURE USE OF WATER POWER IN NEW YORK.

In the foregoing pages we have seen that the Erie Canal was a development from the necessities of commerce, not only for the State of New York, but, as a means of connecting the Atlantic Ocean with the waters of the Great Lakes, for accelerating the industrial development of the Northwestern States. However, in the nineteenth century events move rapidly, and what was true of the Erie Canal thirty to fifty years ago is not necessarily true to-day. Railway systems have now developed to such completeness as to compete successfully with water transportation by a channel of the size of the Erie Canal. The Erie Canal, therefore, has no longer an indispensable place in our transportation system. Apparently it should either be radically enlarged, with entirely new methods of management, or else abandoned in favor of a ship canal along other routes.

During the period covered by the rise and decline of the Erie Canal as the important factor in through transportation between the East and a large portion of the West the economic conditions of the interior portion of New York have entirely changed. Cheap transportation, by way of Erie Canal and the Great Lakes, has led to a phenomenal development of agriculture on the broad plains of Minnesota and the Dakotas, where, by the use of modern agricultural machinery, grain can be raised at a profit at such prices as to drive the New York grain grower from the market. The cheap transportation afforded by the Erie Canal has, therefore, to a considerable degree, led to the passing of supremacy from the hands of the Eastern farmer, a loss which can only be regained by the development to the fullest extent of the manufacturing industries of New York, thus making a home market for farm products that can not be transported a long distance, such as garden truck and small fruits. The people of the State of New York can purchase the Western breadstuffs as cheaply as they can be produced at home, and this condition is likely to continue indefinitely.

The long supremacy of the navigation interests has moreover led to the incorporation in the law, jurisprudence, and public policy of this State of certain rules of action as to the right to use the water of inland streams, which have tended to discourage the full development of manufacturing interests which now appears desirable, although the author views with satisfaction the rapid change of public sentiment now taking place on these questions. That manufacturing industries by water power are rapidly increasing in the State is made sufficiently clear by the following statistics:

According to the United States censuses of 1870 and 1880 the total developed water power of the State of New York was, in 1870, 208,256

horsepower; in 1880, 219,348 horsepower; increase in the ten years, 11,092 horsepower. The increase in ten years of 11,092 horsepower is equivalent to an increase of 5.4 per cent. The United States census of 1890 did not include any statistics of water power, and it is impossible therefore to state definitely the horsepower in that year; still, taking into account the great increase shown by the special investigations on Hudson River in 1895, on Genesee River in 1896, and at Niagara Falls in 1897, and also considering the advances in paper making—a water-power industry—as well as the great development now taking place at Massena, the increase for the whole State from 1880 to 1900 may be estimated at about 120 to 140 per cent. On this basis there will probably be in use in New York State at the close of the nineteenth century a total water power of something like 500,000 gross horsepower. The manufacture of mechanical wood pulp alone consumes about 125,000 gross horsepower. These figures, while very suggestive as to the future, are nevertheless rendered more pertinent by considering that with full development of the water-storage possibilities of the State, as well as the possibilities of power development on Niagara and St. Lawrence rivers, we may hope ultimately to reach a water-power development in the State of New York something like the following:

Possible development of water power in New York.

	Gross horsepower.
Streams tributary to Lake Erie	3,000
Niagara River (in New York State)	350,000
Genesee River and tributaries	65,000
Oswego River and tributaries	40,000
Black River and tributaries	120,000
Other tributaries of Lake Ontario	10,000
St. Lawrence River	400,000
Oswegatchie, Grass, Racket, St. Regis, Salmon, Chateaugay, and other streams tributary to the St. Lawrence	150,000
Saranac, Ausable, Lake George Outlet, and other streams tributary to Lake Champlain	40,000
Hudson River and tributaries, not including Mohawk River	210,000
Mohawk River and tributaries	60,000
Streams tributary to Allegheny River	5,000
Streams tributary to Susquehanna River	25,000
Streams tributary to Delaware River	30,000
Water power of Erie Canal	10,000
Total	1,518,000

But 1,518,000 gross horsepower has an effective productive value in manufacturing of say $100 per horsepower per annum, or the inland waters of this State have an ultimate economic value, when fully developed, of at least $151,800,000 per annum. They may therefore be considered, in producing capacity, substantially equal to the entire agricultural product of the State in 1890, which, according to the United States census of that year, amounted to a total of $161,593,009.

Indeed, taking into account that agricultural values are continually depreciating, and water-power values appreciating, it is probable that ultimately, if New York State agriculture remains on the same basis as at present, the water-power values will considerably exceed the agricultural values. It is probable, however, if the manufacturing industries of this State are ever so far developed as to bring water power into use to the extent of 1,518,000 gross horsepower, that the local demand for agricultural products will have considerably changed the present downward tendency. As an off-hand figure, we may, therefore, place these two values, at some not very distant date, as equal, and aproximating about $200,000,000 per annum.

OBSTRUCTIVE EFFECT OF FRAZIL OR ANCHOR ICE.

A very serious difficulty in operating water powers on many of the more rapid streams of this State is that caused by the formation and agglomeration of frazil and anchor ice, and probably there is no subject in connection with water-power development which presents so many difficulties as this. So far as can be learned, nothing has been done in the State in the way of studying these phenomena, although the water powers on many New York streams are reported as subject to interruption nearly every year on account of the formation of frazil and anchor ice. The way to find a remedy is first to ascertain all that can be learned in regard to the difficulty to be overcome. From this point of view it is deemed proper to include herein a short account of studies of frazil and anchor ice made in the neighboring Dominion of Canada.

Under the direction of John Kennedy, chief engineer of the harbor commissioner's works at Montreal, very extensive studies of the formation of frazil and anchor ice have been made. The terms "frazil" and "anchor ice" have been used synonymously, and are apparently often understood as the French and English words for the same thing, but the following from the Report of the Montreal Flood Commission of 1890 will serve to define the difference. According to this report, frazil is formed over the whole unfrozen surface wherever there is sufficient current or wind agitation to prevent the formation of border ice; whereas the term anchor ice includes only such ice as is found attached to the bottom. Frazil is frequently misused by being made to include ice formed on the bottom, as well as throughout the mass and on the surface of a river, although properly it should be only applied to floating ice. The common theory has been that anchor ice first forms on the bottom, subsequently rising. The Montreal studies, however, show that this is hardly true. At times the whole mass of water from surface to bottom is filled with fine needles which actually form throughout the water mass itself.

As to the remedy, the studies are hardly complete enough to indicate the best course to pursue. As practical hints, it may be stated

that in locating dams on streams specially subject to this difficulty they should be placed with reference to as long a stretch of backwater and as great depth as possible, all the studies thus far made tending to show that the formation is most extensive in shallow, rapid-flowing water. Usually trouble from frazil and anchor ice extends through a period of a day or two; and at very important plants, where even a short interruption would be a serious matter, arrangements may be made for using steam at the head works for keeping the racks open. This plan has been successfully pursued at the waterworks intakes of several of the Great Lake cities. In the case of power plants, where much larger quantities of water are required and the stream flows with greater velocity, the amount of steam required may be found to be very large.[1]

WATER YIELD OF THE SAND AREAS OF LONG ISLAND.

Long Island is about 114 miles in length, with a varying width of from 10 to 20 miles. Its watershed line consists of a regular ridge of low hills running from New York Bay to the eastern extremity of the island. The highest points of this ridge are about 350 to 390 feet above sea level. This ridge, which is believed to be a part of the terminal moraine of the great glacier, consists mainly of compact drift and bowlders, running at times into clay and coarse gravel. The considerable number of small ponds along the ridge evidence the compactness of its surface material. The slopes and spurs of the central ridge run into Long Island Sound on the north, making an irregular shore line, broken into bays and low headlands. On the south side the slopes lose themselves in a grassy plain sloping gently toward the coast. In its widest part it is called the Hempstead Plains, and stretches for a distance of from 5 to 15 miles between the foot of the central ridge and the Atlantic shore, which is very regular in its outer beach line; but an inner and more irregular beach exists, formed by the shallow waters of Jamaica and Hempstead bays. The Atlantic shore does not anywhere touch the slope of the central ridge, but is separated from it by the wide gravelly plain just referred to.

Numerous small brooks originating on the south slopes of the central ridge cross the gravelly plain, delivering their waters to the Atlantic. On the largest of these brooks gristmills were established at an early date, with ponds of from 8 to 40 acres of water surface and from 5 to 9 feet depth of water.

The fall at these dams rarely exceeds 8 feet. The original municipal water supply of the city of Brooklyn, as constructed about 1856 to 1859, had its source in the Hempstead Plains, several of the large brooks flowing from the central ridge to the Atlantic being appropri-

[1] For reference to the literature of frazil and anchor ice see (1) Report of the Montreal Flood Commissioners of 1886; (2) Reports of the Harbor Commissioners of Montreal for the years 1885, 1887, and 1895; (3) Paper on Frazil ice and its nature, and the prevention of its actions in causing floods, by George H. Henshaw, Trans. Can. Soc. C. E., Vol. I, Part I, pp. 1-23; and (4) Paper on the Formation and agglomeration of frazil and anchor ice, by Howard T. Barnes, in Canadian Engineer, Vol. V (May, 1897).

ated for this purpose. A distributing reservoir was established on the central ridge at an elevation of 170 feet above tide, with the water of the brooks forced thereto by pumping. These brooks were all mainly fed by springs delivering directly into their ponds and channels. The length of these water courses from where the water was taken to the summer sources rarely exceeds 4 miles. In the original construction the waters of these ponds were conveyed by small branch conduits to a large main conduit extending from the most easterly pond or reservoir to the pump well at the engine house, which was located at the foot of the ridge on which the Ridgewood distributing reservoir was situated, not far from the east line of the city of Brooklyn. The main conduit was so located that the water flowed to the engine house by gravity. The following are the statistics of the six ponds originally taken for the Brooklyn city supply, the minimum deliveries here given being as ascertained by measurements during the months of September and October, 1856 and 1857. The figures represent the natural delivery of each stream at its lowest stage of water, and do not include any encroachment upon the stored water which each pond retained, when full.

Area of surface, minimum flow, and elevation of overflow of six ponds originally taken for the Brooklyn city supply.

Pond.	Area of surface (acres).	Minimum flow (cubic feet in 24 hours).	Elevation of overflow above tide (feet).
Jamaica	40.00	419,315	7.90
Brookfield	8.75	265,098	15.40
Clear Stream	1.07	100,448	11.50
Valley Stream	17.78	325,291	12.80
Rockville	8.00	353,388	12.60
Hempstead	23.52	1,054,713	10.60

The same streams were measured in October and November, 1851, and the aggregate result then was 3,137,500 cubic feet. With the exception of Clear Stream, they were again measured in October, 1852, the result then being 2,606,300 cubic feet in 24 hours.

According to a survey made by Theodore Weston in the fall and winter of 1859, the drainage area of the streams originally taken for the municipal supply of Brooklyn was found to measure 46.8 square miles, but subsequent measurements have placed it at 49.9, which is the figure now used.[1]

As already stated, the drainage grounds lie mainly on the Hempstead Plains, although a small portion may be considered as lying on the southern slope of the central ridge. The ridge slopes are composed of clay and alluvial earth, with little power of retaining

[1] *As to the difficulty of determining just what the drainage area of any one of these streams actually is, see* I. M. De Varona's History and Description of the Brooklyn Waterworks, 1896.

water. Hempstead Plain, on the other hand, consists of a very uniform deposit of sand and gravel with occasional thin veins of clay; hence Hempstead Plain is largely receptive and retentive of water. The sand and gravel on this plain serves two purposes as regards the rainfall sinking into it: (1) It retains the water, only gradually delivering it to the surface in the valleys of the brooks or on or near the seashore in the form of springs; (2) it filters and purifies it, the gravel and sand performing the function of a natural filter bed. It is considered that but a small portion of the ground water of this gravel plain has been derived from the rainfall of any single year. The greater portion of it is considered to have collected during a series of years. Borings and open wells show that this ground water has a nearly uniform inclination toward the south shore of about 12 feet per mile.

Upon the low ridges lying between the several streams crossing Hempstead Plain the inclination of the ground water varies with the width of the ridge, and is steeper in these parts than on the main slope toward the sea, the resistance of the retaining material there being proportionately less. So long as the slope of the ground water is left undisturbed by pumping, as from a series of wells, the permanent slope of the ground water is determined by the resistance of the material through which it flows. As regards the minimum flow of the streams receiving these underground waters, the longer the time occupied by that portion of the rainfall which sinks into the ground in reaching the outlets the greater will be the minimum flow of the stream as compared with its total flow; on the other hand, the shorter the time the smaller the minimum flow. In the case of the Long Island streams the minimum flows are not very large, a fact which indicates that the permanent regimen of these streams is probably maintained by the accession of the absorbed rainfalls of several years. It follows that so long as the basins are not drawn upon very greatly in excess of their flowage capacity the permanency of Long Island ground water supplies is only moderately affected by variations in the yearly rainfall.[1]

The water supply of the city of Brooklyn has frequently been increased in order to meet the necessities of the constantly increasing population. Additional drainage areas have been taken, and open wells and driven-well systems have been constructed and additional streams and ponds appropriated. At present there are a number of deep open wells, each about 50 feet in diameter, and several driven-well stations. For details of the original conduits, ponds, and streams reference may be made to a description of the Brooklyn waterworks in Water Power of the United States, Tenth Census, 1880, Volume XVII, as well as to the histories of the Brooklyn waterworks.

[1] The foregoing statements relating to the water-yielding properties of the Long Island sands are mostly derived from Kirkwood's History of the Brooklyn Waterworks and Sewers, published in 1867. For a more recent, as well as more extended, discussion of the same subject see De Varona's History and Description of the Brooklyn Waterworks.

In a report on the future extension of the water supply of Brooklyn, by Mr. I. M. De Varona, engineer of the water supply, transmitted to the common council, January 31, 1896, tables are given of the total monthly and average daily quantities of water pumped into the Ridgewood reservoir for the years 1860 to 1895, inclusive.

The accompanying table has been condensed from this report, giving in calendar years the total rainfall upon the watershed and the per cent of this utilized by pumping at Ridgewood. The average yield utilized is also expressed in cubic feet per second per square mile of watershed. This was originally 49.9 square miles, but was increased in 1872, being in subsequent years 52.3 square miles until 1883, when it was increased to 64.6 square miles, and in 1885 to 65.4 square miles. Considerable additions were made in 1891, and from that time on the area is given as 154.1 square miles. In 1860 the rainfall was 37.65 inches, and the total amount of water pumped was equivalent to a depth of 1.44 inches on the watershed, or 3.82 per cent of the total rainfall. In 1896 the total rainfall was 38.82 inches. The amount of water pumped during that year would cover the watershed to a depth of 11 inches, this being over 28 per cent of the total rainfall. The average yield as obtained by pumping was 0.81 cubic foot per second per square mile of watershed.

Total annual rainfall, per cent utilized, and average yield per square mile of watershed of Brooklyn waterworks.

Year.	Rainfall in inches.	Per cent utilized.	Second-feet per square mile.	Year.	Rainfall in inches.	Per cent utilized.	Second-feet per square mile.
1860	37.65	3.82	0.11	1879	39.61	33.40	0.97
1861	45.65	3.92	0.13	1880	40.76	30.23	0.90
1862	38.02	5.73	0.16	1881	39.53	29.42	0.86
1863	32.76	8.39	0.20	1882	39.83	30.73	0.90
1864	32.00	10.53	0.25	1883	37.22	33.05	0.91
1865	46.14	8.39	0.28	1884	45.39	27.89	0.93
1866	51.68	8.88	0.34	1885	36.85	37.94	1.03
1867	54.61	9.39	0.38	1886	51.38	28.32	1.07
1868	38.58	17.29	0.49	1887	45.66	32.59	1.10
1869	43.13	17.20	0.55	1888	48.45	33.19	1.18
1870	39.25	19.82	0.57	1889	56.54	29.54	1.23
1871	51.26	15.78	0.60	1890	52.15	33.90	1.30
1872	39.75	23.47	0.67	1891	39.18	44.82	1.29
1873	47.99	20.88	0.74	1892	37.75	24.53	0.68
1874	45.83	21.49	0.73	1893	39.62	26.27	0.77
1875	40.90	26.89	0.81	1894	36.88	26.33	0.72
1876	41.77	27.08	0.83	1895	35.64	28.98	0.76
1877	40.18	30.29	0.90	1896	38.82	28.31	0.81
1878	48.66	25.15	0.90				

Generally the Brooklyn waterworks have not been so designed as to furnish records of the quantity drawn from these several different sources. There are also no records of the heights of the ground water at different points in the drainage area. If such were to be kept for a series of years, the records of the Brooklyn waterworks would possess a value not easily estimated. They would give a far more positive indication of the amount of water that can be drawn from such sandy areas than can now be gained from them. A few tests, however, of some of the driven-well plants have been made in the last few years.

At a test of the old driven-well plant at Spring Creek, made from October 22 to November 20, 1894, water was pumped at an average rate of 4,091,551 gallons in 24 hours. The elevation of the underside of the discharge valve of the pump was 12.3 feet above datum. On October 22, at the beginning of the tests, the average elevation of the water in the wells was 4 feet below datum. The quantity pumped in 24 hours, on October 22, was 4,488,275 gallons. On November 20, the date of the conclusion of the test, the elevation of water in wells was 7.7 feet below datum, and the quantity pumped on that day in 24 hours was 4,112,663 gallons. The total quantity pumped during the entire period from October 22 to November 20 was 122,746,525 gallons. The taking of this quantity of water from the wells resulted, therefore, in lowering the ground water a total of 3.7 feet.

A new driven-well plant at Watts Pond was subjected to a test of capacity extending continuously from January 3 to February 2, inclusive. In 1895 a rather extended series of tests were made of a number of the wells of the Brooklyn water supply in order to determine the yield as well as the extent of the underground supply. The following particulars of these tests have been derived from Mr. De Varona's report, as contained in the annual report of the commissioner of city works for the year 1895.[1]

The flowing wells at Jameco were tested from January 3 to 14, inclusive. During this period the wells were operated singly and in groups of 2, 3, and 4, in all possible combinations, and observations were taken to determine the elevation of the ground water. Upon completion of the tests a series of observations was taken, extending to January 30, to determine the normal water level. The results of these observations may be found in detail in De Varona's report. It was shown that the average yield from one well alone was only 1,000,000 gallons daily, decreasing pro rata up to a total yield of 3,500,000 gallons daily when four wells were in operation. The lowering of the ground water was approximately 5 feet when pumping 1,000,000 gallons, increasing up to approximately 10 feet when pumping at the full capacity developed of 3,500,000 gallons. In this connection it is stated that the water in these test wells is found to rise

[1] For a full account of the Brooklyn waterworks well systems see De Varona's History, etc.

and fall directly with the tide, thus rendering it difficult to state with accuracy the full effect of the pumping on the lowering of the water. To determine this point fully, De Varona states, would require a more prolonged series of observations than it was possible to make in 1895.

Another test was made at Jameco from December 9 to 20, 1895, inclusive. Between this date and the end of the previous tests an additional well had been sunk at Jameco to the depth of 160 feet. The average daily yield shown during the second test was, approximately, 1,000,000 gallons for a single well, with a proportionate increase for each well connected, the yield for five wells being, approximately, 5,000,000 gallons in 24 hours. The lowering of the water during those tests amounted to slightly over 14 feet at Jameco while pumping the 5,000,000 gallons daily from the five wells. The total amount of water pumped during the test was 61,239,555 gallons. The greatest lowering of the underground water level occurred at test well No. 8, where it amounted to 15.23 feet. At that time, when the water at Jameco was at its lowest level, the fall between test well No. 8 and test well No. 11 was 9.9 feet. The normal water level was not restored until twelve days after the tests had ceased.

The results obtained early in 1895 from the test made at Jameco of supplies from deep wells seemed to warrant further investigations as to the possibility of water from deep wells, and the report states that they have been carried on during the year. A series of test wells were driven, extending from the foot of the hill at Ridgewood reservoir to Forest Stream pumping station, each well being carried to a depth sufficient to determine the possibility of obtaining a deep supply from that point. The number of those wells sunk during that year was twelve, and the records of the strata passed through are given in Bulletin No. 138, referred to in the footnote.[1]

Returning to the table on page 194, it may be stated that the tributary drainage area in 1875, the first year for which statistics are given in the table, was 52.3 square miles. The drainage area remained at this figure until January, 1881, in which month, by the bringing of the Springfield pumping station into use, it was increased to 59.4 square miles. In the water year of 1875, with a total rainfall of 41.6 inches, the water utilized amounted to 10.78 inches, or to an average of 513,165 gallons per square mile per day, or to 0.79 of a cubic foot per square mile per second. In the water year of 1880, with a total rainfall of 40.04 inches, the water utilized amounted to 12.37 inches on the watershed, or to 587,568 gallons per square mile per day, or to 0.91 of a cubic foot per square mile per second. In 1881, with a rainfall of 41.52 inches, the total utilization of water amounted to 11.64 inches on the watershed, or to 554,473 gallons per square mile per day, or to 0.86 of a cubic foot per square mile per second. This drop in

[1] For the particulars of the geology of several of the Brooklyn waterworks wells, of which tests were made in 1895, see Artesian-well prospects in the Atlantic coastal plain region, by N. H. Darton: Bull. U. S. Geol. Survey No. 138, 1896, pp. 23-37.

the unit of utilization merely shows the effect of the increase in the area of the watershed.

The tributary watershed remained at 59.4 square miles until August, 1883, in which month the Spring Creek and Baisley's driven-well stations were started. From this date the tributary drainage area is taken at 64.6 square miles. Spring Creek and Baisley's stations marked the beginning of the Brooklyn driven-well system. In the water year of 1884, with a total rainfall of 43.44 inches, the utilization was 12.53 inches, amounting to 594,992 gallons per square mile per day, or to 0.92 of a cubic foot per square mile per second.

In May, 1885, the Forest Stream and Clear Stream driven-well stations were started, thereby increasing the tributary drainage area to 65.4 square miles. In the water year of 1886, with a total rainfall of 50.43 inches, the water utilized amounted to 14.40 inches, equivalent to 685,521 gallons per square mile per day, or to 1.06 cubic feet per square mile per second.

The drainage area remained 65.4 square miles until June, 1890, when it was increased to 65.6 square miles by the addition of the Jameco Park driven-well station. In the water year 1891, with a total rainfall of 40.34 inches, the water utilized amounted to 18.48 inches on the watershed, equivalent to 879,811 gallons per square mile per day, or to 1.35 cubic feet per square mile per second.

Large extensions of the works were made in 1890 and 1891, so that with the beginning of pumping at Millburn on December 17, 1891, the tributary drainage area may be considered as increased from 65.6 to 154.1 square miles, an increase of 88.5 square miles. In the calendar year 1892, with a rainfall of 37.75 inches, the water drawn from the original watershed of 65.6 square miles amounted to 16.81 inches on the watershed, equivalent to 800,191 gallons per square mile, or to 1.24 cubic feet per second per square mile. The water drawn from the new watershed of 88.5 square miles that year amounted to 3.67 inches on the watershed, equivalent to 174,776 gallons per square mile per day, or to 0.27 of a cubic foot per square mile per second. In 1895, with a total rainfall of 35.64 inches, the original watershed of 65.6 square miles yielded 12.62 inches on the watershed, equivalent to 600,723 gallons per square mile per day, or to 0.93 of a cubic foot per square mile per second. The new watershed of 88.5 square miles furnished in that year 8.64 inches on the watershed, equivalent to 411,558 gallons per square mile per day, or to 0.64 of a cubic foot per square mile per second.

Summarizing the information in regard to the water yield of the sand plains of Long Island, it may be stated that the available data indicate a large yield. The streams of eastern New York can not be relied upon in their natural condition to yield more than about 0.3 of a cubic foot per square mile per second, while with an ordinary development of storage the limit may be usually placed at from 0.7

to 0.8 of a cubic foot per square mile per second, or at any rate at not much exceeding 1 foot per square mile per second. The sand deposits of Long Island may, therefore, be considered as great natural reservoirs from which, with proper development, large water supplies may be drawn, the same as from reservoirs artificially created on the earth's surface, these natural underground reservoirs possessing the advantage of furnishing a filtered water of high purity.

INDEX TO PAPERS NOS. 24 AND 25.

Page.

Adirondack Plateau, water yield of..... 16-18
Allegheny Mountains, water yield of 16
Allegheny River, hydrography of..... 44
Anchor ice, obstruction from..... 14,190-191
Ausable River, fall of 32
Battenkill River, hydrography of..... 40-41
power of 41
Black River, hydrography of..... 29-30,96-97
Black River Canal, cost and revenues of. 154-158
length and capacity of 158
Black Rock, water power at 178-179
Brooklyn, water supply of 15,43,47,191-195
Buffalo Creek, course of 24
Canadaway Creek, power of..... 24
Canals, construction of 145-150
cost and revenues of 154-155
improvement of 155-157
transportation by 150-154
water supply of 158-161,164-166,173-178
See Ship canals.
Canaseraga Creek, course of 25
Catskill Mountains, water yield of 16
Cayuga and Seneca Canal, construction of..... 149
cost and revenues of 154
Cayuga Lake, drainage area of 28
Champlain Canal, construction of..... 148-149
cost and revenues of 154-157
length and capacity of 157,158
water supply of 159-161
Chateaugay River, course of 30
Chemung Canal, construction of 149
cost and revenues of 154-155
Chemung River, drainage area of..... 88
floods in..... 12,87-90
Chenango Canal, construction of..... 149
cost and revenues of 154-155,157-158
length and capacity of 157-158,161-165
loss of water in..... 174-175
Chenango River, course of..... 45
Chicago drainage canal, water diverted through..... 62
Clyde River, drainage area of 28
Cohocton River, course of..... 45
Cohoes, water power at 35,185-186
Crooked Lake Canal, construction of.... 149
cost and revenues of..... 154-155
Croton River, hydrography of... 12,33,82-86,98
storage on..... 86-87
Deforestation, effects of 11,12,14,16-17
Delaware River, hydrography of..... 46-47
Des Plaines River, hydrography of. 54-55,64-65

Page.

East Canada Creek, hydrography of..... 36-37
power of 37-38
Eaton Brook, hydrography of..... 67,68
Erie Canal, construction of..... 13,147-150
cost and revenues of 154,157
improvement of 155-156,169-171
length and capacity of..... 157,158,161-165
transportation on 13-14,150-154
water power of..... 178-184
water supply of..... 158-161,164-166,173-178
Fish Creek, fall of..... 36
Fish Creek, power of..... 41
Floods, occurrence of 12,72-73,87
Forests, influence of..... 16-18
Frazil, obstruction from..... 14,190-191
Fredonia, stream measurements near.... 95
Genesee River, flood on..... 112-113
hydrography of .. 11,25-27,57,58,70-75,90-92
power of 109,123-125
storage on 12,109-125
tributaries of 25-26
Genesee Valley Canal, cost and revenues of..... 154-155
Grass River, course of 30
Great Lakes, discharge of 58
drainage area of..... 48
precipitation on drainage area of.. 49-53,58
Hemlock Lake, hydrography of. 12,75-77,92-93
Honeoye Creek, power of..... 26
Hoosic River, hydrography of..... 40
power of..... 40
storage on 40
Hudson River, commerce of..... 144-145
hydrography of..... 33-34, 79-82,97-98
logging on 14
precipitation on drainage area of.... 133
storage on 12,125-134
tributaries of..... 33-43
Jameco, wells at..... 195-196
Kayaderosseras Creek, power of..... 41
Lake Champlain, streams tributary to... 31-33
Lake Erie, altitude of..... 59
Lake Ontario, drainage area of..... 65
Little Falls, water power at..... 35
Lockport, water power at..... 179-182,186
Long Island, hydrography of.. 14-15,47,191-198
Low-water flow of streams..... 90-99
Lumber, transportation of..... 14
Madison Brook, hydrography of..... 67,68,69
Massena, power development at.... 12,143-144
Mechanicville, stream measurements at. 81-82
Medina, water power at..... 182-184

Page.
Mohawk River, hydrography of 35, 97
tributaries of 35–40
Morris Run, discharge of 93–94
Mount Morris, proposed reservoir at 110, 113–114, 117, 122–125
stream measurements at 58, 72–73, 91
Muskingum River, hydrography of 55–57
Neversink Creek, course of 46
Newell, F. H., letters of transmittal by .. 7, 107
Niagara Falls, power development at 12, 135-143, 186
Niagara River, hydrography of 11, 24–25, 48, 59–63
power development on 12, 135–143
Oatka Creek, hydrography of 69–70, 90
Oneida River, drainage area of 29
Oriskany Creek, course of 40
Oswegatchie River, course of 30
Oswego, water power at 184–185
Oswego Canal, construction of 149
cost and revenues of 154, 157
length and capacity of 157
Oswego River, hydrography of 27–29, 96
power of 29
Owasco Lake, drainage area of 28
Ownership of water 10, 14–15, 186–188
Paper, manufacture of 14
Pepacton River, course of 46
Portage, proposed reservoir at 114–125
stream measurements at 119–121
Precipitation, amount of 19–21
Price of water power 184–186
Raquette River, course of 30
Rochester, stream measurements at 72, 76–77, 91, 119
water power at 186
Rome, water supply of 35

Page.
Sacundaga River, hydrography of 42
storage on 128, 131
St. Lawrence River, hydrography of 65–66
power development on 12, 143–144
tributaries of 24–31
St. Regis River, course of 30
Saranac River, fall of 32
Sauquoit Creek, course of 40
Schoharie Creek, hydrography of 35–36
Schroon River, hydrography of 43
storage on 129
Seneca Lock Navigation Company, operations of 147
Seneca River, drainage area of 28
Ship canals, proposed routes for 166–173
Skaneateles Lake, hydrography of 77–78, 96
Spruce Creek, fall of 37
Storage reservoirs, advantages of 12, 111-112, 132–134
canals supplied by 164–166
capacity and cost of 116–117, 130–131
sites for 109–111, 113–125
Susquehanna River, hydrography of 44–46
Syracuse, stream measurements at 77–78
Temperature, variations in 18–19
Tioga River, course of 45
Tonawanda Creek, course of 24
Topography, character of 16–18, 21–23
Upper Mississippi reservoirs, discharge of 53–54
Warsaw, stream measurements at 93–94
West Branch of Canadaway Creek, discharge of 94–95
West Canada Creek, hydrography of 38–39
power of 39
Western Inland Lock Navigation Company, operations of 146

Stanford University Libraries

To avoid fine, this book should be returned on
or before the date last stamped below

25M—7-42

659116